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CHAPTER 3
BRIDGE DESIGN GUIDELINES

3.1 DESIGN CRITERIA

3.1.1 Design Specifications

3.1.1.1 All designs for highway bridges shall be performed in accordance with the latest edition of the following specifications, with current interims as of the date of the design.

This Bridge Manual and all references to the documents provided below are based upon the indicated edition dates.

1. American Association of State Highway and Transportation Officials (AASHTO), LRFD Bridge Design Specifications, 9th Edition (Referred to as *AASHTO LRFD* throughout the remainder of Part I of this Bridge Manual).
2. The Commonwealth of Massachusetts, Department of Transportation, Standard Specifications for Highways and Bridges, 2023 Edition.
3. AASHTO/AWS Bridge Welding Code (AASHTO/AWS D1.5), 2020.
4. American Association of State Highway and Transportation Officials (AASHTO), LRFD Bridge Construction Specifications, 4th Edition.
5. American Association of State Highway and Transportation Officials (AASHTO), Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with Interims through 2022.

Note: Specific references to these documents are provided in numerous locations of the Bridge Manual (Parts I, II and III) and they are valid at the time of writing of this document. However, AASHTO periodically revises its provisions and MassDOT does not necessarily issue revisions to its Bridge Manual concurrently. It is also possible even when a revised Bridge Manual is issued references may be missed or overlooked during the editing. It is the responsibility of the Designer to review all references and apply them appropriately. Any errors or confusion in this regard shall be brought to the attention of MassDOT.

3.1.1.2 All designs for pedestrian bridges shall be performed in accordance with the latest edition of the American Association of State Highway and Transportation Officials (AASHTO), *LRFD Guide Specification for the Design of Pedestrian Bridges*.

3.1.1.3 All designs for railroad bridges shall be performed in accordance with the latest edition of the American Railway Engineering and Maintenance-of-Way Association (AREMA), *Manual for Railway Engineering*.

3.1.2 Critical and Essential Bridges

For the design of bridges in Massachusetts, Critical and Essential Bridges are defined as those bridges that are:

1. On or over the following National Highway System (NHS) routes:
   a. Eisenhower Interstate System.

b. Only those routes that are defined as OTHER NHS Routes and are denoted by red lines on the FHWA HEPGIS website map. Designers are cautioned not to assume that the term OTHER means everything else that is on the NHS. OTHER NHS Routes is a specific category of NHS routes that are separate from and are not to be confused with Intermodal Connectors, Unbuilt NHS Routes and MAP-21 NHS Principal Arterials. These last three categories of NHS routes are not considered Critical and Essential Bridges for the purposes of this Bridge Manual unless they meet the definition set forth in bullet 2 below.

c. All STRAHNET Routes and Connectors including Non-Interstate STRAHNET Routes, Major STRAHNET Connector, and Intermodal/STRAHNET Connector.

2. On designated emergency evacuations routes.

Other bridges may be designated as Critical/Essential by local agencies if they need to be operational after a natural disaster or other event. MassDOT does not make any performance distinction between Critical and Essential bridges. Interactive maps of the National Highway System may be found on the following website: https://hepgis.fhwa.dot.gov/fhwagis/

For the purposes of seismic design the term Critical/Essential, used here, shall be analogous to Critical and Recovery as used in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

3.1.3 Live Load

3.1.3.1 The minimum AASHTO design live load for all new or replacement highway bridges, culverts, soil-corrugated metal structure interaction systems, and walls shall be full HL-93 loading, unless specified otherwise.

3.1.3.2 Existing highway bridges that are being rehabilitated, including superstructure replacements, should be upgraded consistent with the provisions of Paragraph 2.1.2.1 of Part I of this Bridge Manual.

3.1.3.3 Deck Replacements, if practical, should meet the requirements of Paragraph 3.1.3.1 above. However, when this is not practical, the design live load shall be determined using the methodology of Paragraph 2.4.2.8 of Part I of this Bridge Manual. Typically, substructures need not be evaluated. In cases where MassDOT determines that the substructure needs to be evaluated, it shall be done in accordance with Paragraph 2.4.2.8 of Part I of this Bridge Manual.

3.1.3.4 Superstructure Repair, Substructure Repair, Joint Replacement, and other Bridge Preservation Projects: the design of the component being repaired shall be to the original design live load or the Posting Vehicles, which ever is greater, as defined in Chapter 7 of Part I of this Bridge Manual. Repairing a single component will not require that the entire bridge be upgraded to these loads.

3.1.3.5 Historic structures that are being rehabilitated may be exempted from complying with Paragraph 3.1.3.2 if the structure's inventory rating can be upgraded to meet the anticipated truck traffic loadings. These exemptions shall require prior written approval from MassDOT. Substructures shall be in accordance with Paragraph 2.4.2.8 of Part I of this Bridge Manual.

3.1.4 Design Methods

3.1.4.1 All new bridges and complete bridge replacements shall be designed using the Load and Resistance Factor Design (LRFD) method.

3.1.4.2 Bridge projects, such as Deck Replacement, Bridge Superstructure Repair, Bridge Substructure Repair, Joint Replacement, Painting, and other Bridge Preservation or Repair Projects, are primarily maintenance projects and need not bring the entire bridge up to current AASHTO design code and Bridge Manual standards. Therefore, the 17th edition of the *AASHTO Standard Specifications for Highway Bridges* may be used in place of the LRFD method. Furthermore, the minimum design live loading to be used for design should be either be the original design truck or the Posting Vehicles as defined in Paragraph 7.2.4.1B of Part I of this Bridge Manual, whichever is higher. However, a Deck Replacement Project, because the entire deck is being replaced, affords the Designer the ability to potentially improve the load carrying capacity of the bridge (if needed) and to upgrade the railing/barrier to current standards.

3.1.4.3 To verify that the design will also provide adequate load carrying capacity for the Massachusetts posting vehicles, preliminary load rating calculations shall be performed in accordance with Chapter 7 of Part I of this Bridge Manual as part of the design process and will be submitted with the structural submittals as described in Section 4.3 of Part I of this Bridge Manual. These calculations along with the Rating Summary shall be reviewed as part of the design review process and shall be returned to the Designer for corrections and revisions to the Structural Design Calculations, if needed. The actual rating report, as described in Chapter 7 of Part I of this Bridge Manual, need not be submitted until the bridge has been constructed, the Initial Inventory Inspection performed, and any design changes made during construction have been rated and incorporated into the final rating report.

3.1.4.4 *AASHTO LRFD* use a Load Modifier, η_i consisting of the product of three factors: Ductility, Redundancy and Importance. The values of the factors to be used shall be as follows:

- Ductility: use η_D=1.05 for nonductile components and connections and 1.0 for all others.
- Redundancy: use η_R=1.05 for nonredundant members and 1.0 for all others.
- Importance: use η_I=1.05 for critical or essential bridges and 1.0 for all others.

3.1.5 Design Calculations and Software

3.1.5.1 Design Calculations. All MassDOT bridge designs shall be based on two sets of calculations: the Design Set of Calculations and an Independent Check Set of Calculations. Each Set of Calculations shall be performed by an engineer working independently and without collaboration with the other. The purpose of these dual calculations is to allow the two engineers to develop their own assumptions and approaches regarding: loads, such as dead loads and live loads, and their distribution throughout the bridge; the bridge's behavior under these loads; and the analytical model, including computer model, used to analyze the bridge. At the conclusion of these calculations, the engineers shall compare the results and any discrepancies that affect member capacity to demand ratios shall be reconciled. The above is not intended to be a comprehensive listing, as the complexity of the design will dictate the items necessary to be reconciled. When this global review is performed, each Set of Calculations need not be checked additionally for mathematical accuracy by another engineer, as any errors will become evident in the final reconciliation. Wherever possible, when calculations are based upon software results, the two sets of calculations should use different applicable software packages.

3.1.5.2 Software. In order to verify program compliance, software used by Designers must be able to replicate the results of designs performed using the software MassDOT uses. Portions of programs not giving similar results will require hand computations to demonstrate conformance.

MassDOT currently utilizes AASHTOWare™ Bridge Design as the standard software for the LRFD design of the following structure types:

- Reinforced concrete frames
- Reinforced concrete tee beams, slabs and I-beams
- Prestressed concrete deck and box beams, I-beams, NEBT beams, NEXT beams, and NEDBT beams
- Steel rolled beams (including cover plates)
- Steel welded straight and curved plate I-girders (including hybrid)

A 1D line girder analysis should be used whenever possible. The request to use a more refined method of analysis (2D or 3D) needs to include justification as to why a 1D analysis is insufficient and requires prior written approval from the State Bridge Engineer. If a more refined method is approved, the Designer will be required to provide a table on the Construction Drawings of dead load force effects and coefficients of live load distribution for each primary member, as required by *AASHTO LRFD* Article 4.6.3.1. Note, that if the Designer includes diaphragms or cross frames in the refined analysis model, they are to be considered primary members and need to be included in the tables of dead load force effects and live load distribution coefficients.

Approval is not required for integral abutment bridges which fall outside of the parameters of the Simplified Method (see Paragraph 3.10.11.1), however this analysis is limited to the effects upon the piles as a result of the soil and thermal movement (see Paragraph 3.10.11.5) and not the design of the superstructure.

3.1.6 Earth Pressure Computations

Earth pressure coefficient estimates are dependent on the magnitude and direction of wall movement. Unless documented otherwise in the approved Geotechnical Report, the following earth pressure coefficients shall be used in design:

- Cantilever walls not founded on rock or piles that are greater than 5' in height or any spread footing-supported gravity wall shall use K_a.
- Cantilever walls not founded on rock or piles that are less or equal to 5' in height shall use $0.5(K_o + K_a)$.
- Counterfort walls, cantilever walls of any height, or gravity walls that are founded on rock or piles shall use K_o.

  Where:

  K_a = Active earth pressure coefficient;

  K_o = At-rest earth pressure coefficient;

Active earth pressure coefficients (K_a) shall be estimated using Coulomb Theory. Passive earth pressure coefficients (K_p) shall be estimated using Rankine or Log Spiral Theory. Current MassDOT practice is to use the unit earth weight of 120 pcf in the calculation of earth pressures where more specific data is not available.

The earth pressure exerted against integral abutments shall be estimated in accordance with Section 3.10 of this Chapter.

3.1.7 Bridge Railings/Barriers

3.1.7.1 MASH Implementation. The AASHTO/FHWA Joint Implementation Agreement for the AASHTO Manual for Assessing Safety Hardware, 2015, requires that bridge projects advertised for construction after December 31, 2019 must only specify bridge railing/barriers and transitions that have been evaluated using the 2015 edition of MASH. In order to comply with this agreement, MassDOT is undertaking a program to re-evaluate and/or crash test the standard MassDOT railings/barriers for compliance with MASH. As each railing/barrier is re-evaluated, it will be given a new Test Level designation that starts with the letter "M" to identify it as being MASH compliant and to distinguish it from an NCHRP 350 compliant railing Test Level. For example, a railing compliant with MASH TL-4 would be designated MTL-4. Although the agreement only affects bridge projects on the NHS, MassDOT will transition to MASH compliant railings/barriers for all bridge projects.

3.1.7.2 The standard MassDOT railings/barriers detailed in Chapter 12 of Part II of this Bridge Manual shall be used in accordance with the Table 3.1.7-1 below.

3.1.7.3 Railings/barriers other than the ones detailed in Chapter 12 of Part II of this Bridge Manual, may be used provided that the use of a non-standard MassDOT railing/barrier can be justified and that they have either been:

1. Crash tested in accordance with and have passed the requirements of the AASHTO Manual for Assessing Safety Hardware (MASH) at a facility that specializes in the crash testing of highway safety appurtenances, or
2. With prior approval by MassDOT, have undergone a computerized crash simulation at a facility that has been approved by FHWA to perform such computer simulations in accordance with the requirements of MASH and the simulation results indicate that the railing/barrier would pass the requirements of MASH.

Railings/barriers that have not been crash tested, have not received approval from FHWA for use on the NHS, or have not undergone a crash test simulation shall not be used on any MassDOT bridge project.

Table 3.1.7-1: Standard MassDOT Railings/Barriers Test Level and Use

Railing/Barrier	Test Level	To Be Used	Application Notes
CT-MTL2	Crash Simulated MASH TL-2	Non-NHS highways only with design speeds not exceeding 45 MPH	Off system bridges w/ or w/out pedestrians; no protective screen or snow screen is required.
S3-MTL4	MASH TL-4	NHS and Non-NHS highways, except limited access highways and their ramps	W/ or w/out pedestrians; must be used with Type I screen. No screen is required on bridges over water or terrain without transportation facilities.

Table 3.1.7-1 (Continued): Standard MassDOT Railings/Barriers Test Level and Use

Railing/Barrier	Test Level	To Be Used	Application Notes
CP-MTL3	MASH TL-3	NHS and Non-NHS highways, except limited access highways and their ramps	W/ or w/out pedestrians, mainly urban bridges and bridges over RR and all structures over electrified AMTRAK rail lines; must be used with either Type II screen or hand rail when pedestrians are allowed on the bridge or with a 4' high snow fence when pedestrians are not allowed on the bridge. No screens are required on bridges over water or terrain without transportation facilities.
CF-MTL5	MASH TL-5	NHS and Non-NHS limited access highways and their ramps	All Interstate and limited access state highway bridges; must be used with 3' high snow fence. No screen required on bridges over water or terrain without transportation facilities.
CM-MTL3	Crash Simulated MASH TL-3	NHS and Non-NHS highways, except limited access highways and their ramps	Curb mounted application: For bridges w/ or w/out pedestrians to protect a non-crash tested ornamental or historic that is being used along the bridge fascia for aesthetic purposes.

3.1.7.4 WARNING. The geometry of the impact face of a railing or barrier is critical to its safe performance in an actual crash. Therefore, Designers are prohibited from altering or attaching anything to the impact face of a railing or barrier that has been crash tested and found to meet the performance requirements of either NCHRP 350 or MASH. If a standard crash tested railing or barrier cannot be used without modifications, the Designer shall confer with the Bridge Section to receive guidance on how to proceed.

3.1.7.5 Steel reinforcement for the deck slab overhangs shall be as per Chapter 12 of Part II of this Bridge Manual. If the deck slab overhang exceeds the limits specified in the design tables of Chapter 7 of Part II of this Bridge Manual, the Designer shall design the deck reinforcement in accordance with Section 13 of the *AASHTO LRFD* for the given test level of the railing/barrier system.

3.1.7.6 In cases where railings/barriers are mounted on top of U-wingwalls or retaining walls, the wall's stability and its stem design shall be as per Subsection 3.3.2.

3.1.7.7 Noise Walls. There are locations where a noise wall must be carried over a bridge. The noise wall may be attached to the back of the bridge barrier, however, because in this application the noise wall is potentially subject to vehicular impacts, the Designer should use a crash tested, light weight, noise wall. The Designer should also be careful to evaluate the load effects of the noise barrier, including wind load and dead loads, on the barrier system. To do this, the Designer should calculate how much of the bridge barrier's reinforcement steel area connecting it to the deck will be taken up by the noise wall load effects. If the amount of bridge barrier reinforcement steel area to be used by the noise wall load effects is 15% of the total reinforcement steel area or less, then the noise barrier may be attached to the back of the bridge barrier. If it is more, then the Designer must seek an alternate attachment for the noise wall independent of the bridge barrier, such as an extension of the deck, although the noise wall may still be braced against the bridge barrier.

3.1.8 Temperature

3.1.8.1 Uniform Temperature. Stresses and movements due to uniform thermal changes shall be calculated in accordance with the *AASHTO LRFD* for the Cold Climate temperature range using the following procedure, which is based upon AASHTO's Procedure A.

3.1.8.2 Since bridge members can be set at different ambient temperatures, the assumed ambient temperature for a temperature rise is different from that used for the temperature fall in order to maximize the range of one-way thermal movements to be used in design. Note that the following temperature rise and fall incorporate the *AASHTO LRFD* 65% factor that is used to calculate the shear deformation temperatures according to *AASHTO LRFD*. Therefore, it shall not be applied to these temperature rise and fall.

The maximum one-way thermal movement, δ_T, for the design of structural components shall be:

$$\delta_T = L\alpha\Delta T$$

Where:

L = the length of member from the point of assumed zero movement to the point where movement is to be calculated (in);

α = Coefficient of Thermal Expansion of member material:

- 0.0000065 for structural steel;
- 0.0000055 for concrete;

ΔT = for Structural Steel Members:

- 70°F temperature rise (from an assumed ambient temperature of 50°F)
- 100°F temperature fall (from an assumed ambient temperature of 70°F)

ΔT = for Concrete Members:

- 30°F temperature rise (from an assumed ambient temperature of 50°F)
- 70°F temperature fall (from an assumed ambient temperature of 70°F)

3.1.8.3 Selecting Bridge Joint System. MassDOT practice is to use the "floating" bridge concept, where there is no defined fixed bearing. Thus, for those bridges designed in accordance with this concept, the point of assumed zero movement shall be taken as the midpoint of the bridge beam, even when it is continuous over a pier. This length shall be used with temperature ranges specified above to calculate the anticipated maximum one-way thermal movement at the end of the bridge to be used to select the joint type. With the increase of the permissible one-way thermal movement for the Saw Cut joint to ¾", most typical MassDOT bridges, including Integral Abutment bridges, would utilize that type of joint. If the maximum one-way thermal movement exceeds ¾", then the Designer shall select a joint type appropriate for the bridge type and geometry. For determining the one-way thermal movement for sawcut or asphaltic bridge joints do not apply the *AASHTO LRFD* load factor of 1.2, because it has already been taken into account in the standard MassDOT details of Part II. However, it should be applied for sizing all other joint types.

If the bridge design requires that a defined fixed bearing be provided, then that bearing will be used as the point of zero movement. Continuous beam bridges with multiple fixed bearings along the length of the beam will require an equilibrium analysis to determine the thermal forces and displacements at each substructure unit.

3.1.8.4 Temperature Gradient. The effects of a thermal gradient need not be considered for typical steel or concrete girder bridges with concrete or timber decks, for timber bridges, or for solid slab and deck beam bridges, as detailed in Parts II and III of this Bridge Manual.

3.1.9 Reinforcing Steel

All reinforcing steel shall conform to the requirements of AASHTO M31 Grade 60, unless approved by the State Bridge Engineer. All reinforcing steel in superstructure elements shall be coated. Refer to Paragraph 3.3.1.6 for coated bar requirements in substructures.

Typical lap lengths are provided in Paragraph 4.2.2.3 of Part I of this Bridge Manual and are based upon the *AASHTO LRFD*, and modified by the following: a bar size factor of 0.8 for #6 and smaller bars, epoxy coated bars, a concrete strength of 5.0 ksi, a top cover of 2”, and a bottom cover of 1.5”.

Please note that the bar size factor is not included in the *AASHTO LRFD* provisions due to the desire to generalize the equations for concrete strengths greater than 10.0 ksi, but is present in ACI 318-11, from which the AASHTO provisions were adopted. It is felt that the exclusion of this factor penalizes concrete deck designs, which predominantly use #4 and #5 bars and rarely utilize concrete strengths greater than 10.0 ksi (refer to *AASHTO LRFD* C5.10.8.2 and ACI 318-11). In cases where reinforcing bars are greater than #6 and/or concrete strengths are greater than 10.0 ksi, the Designer shall calculate the required lap lengths based upon the current *AASHTO LRFD* requirements without the small bar factor modification. Note that this modification only applied to the tension development length and does not apply to hooks in tension.

The lap lengths for 180° hoops are intended for closure pours for accelerated bridge construction only. The lap is the *AASHTO LRFD* basic hook development length with the required modifications. It is MassDOT’s opinion that these hoops, with the inclusion of longitudinal steel through them, are sufficient to develop deck reinforcing. Refer to Part III of this Bridge Manual for details. Refer to the *AASHTO LRFD Guide Specifications for Accelerated Bridge Construction* for additional information.

Designers should use the bar size factor, where applicable, for bar laps in other structural elements, for example stirrups in prestressed concrete beams.

3.2 BRIDGE FOUNDATIONS

3.2.1 General

The recommendations made in the Geotechnical Report shall form the basis for the selection and design of the foundations of the bridge structure. In addition to recommending the foundation type, this report also provides the site-specific design parameters, such as soil resistance, on which the foundation design will be based. Pertinent recommendations from the Geotechnical Report regarding design and/or construction shall be included on the Construction Drawings and in the Special Provisions.

3.2.2 Pile Foundations

3.2.2.1 Pile foundations shall be designed in accordance with the provisions of the *AASHTO LRFD*. The Design Factored Resistance of piles shall be the lesser of the Factored Geotechnical Pile Resistance and the Factored Structural Pile Resistance.

The Design Factored Resistance of the pile shall be greater than the combined effect of the factored loading for each applicable load combination.

3.2.2.2 The pile length estimated by design should be adequate to develop the Nominal Resistance required by all limit states as well as the minimum penetration required for lateral stability, uplift, downdrag, scour, settlement, etc.

3.2.2.3 The additional following criteria shall be used as required:

1. Maximum batter on any pile shall be 1:3. When concrete piles are driven in clay, the maximum batter shall be 1:4.
2. The Geotechnical Report should recommend values for Lateral Resistance provided by vertical or battered piles. The geotechnical analysis, relating lateral resistance to deflection, should be performed based on unfactored loads.
3. Maximum spacing of piles shall be 10 feet on center; minimum spacing shall be 2.5 times the pile diameter, unless an alternate design is performed by the Designer and has been reviewed and approved by MassDOT.
4. Minimum distance from edge of footing to perimeter of pile shall be a minimum of 9 inches.
5. The center of gravity of the pile layout shall coincide as nearly as practical with the resultant center of load for the critical cases of loading.
6. Pile layouts of piers with continuous footings shall show a uniform distribution of piles. Exterior piles on the sides and ends of pier footings may be battered if required by design.
7. Steel pile-supported foundation design shall consider that piles may be subject to corrosion, particularly in fill soils, acidic soils (soils with low pH), and marine environments. Where warranted, a field electric resistivity survey, or resistivity testing and pH testing of soil and groundwater samples should be used to evaluate the corrosion potential. Steel piles subject to corrosion shall be designed with appropriate thickness deductions from the exposed surfaces of the pile and/or shall be protected with a coating that has good dielectric strength, is resistant to abrasive forces during driving, and has a proven service record in the type of corrosive environment anticipated. Protective coating options include electrostatically applied epoxies, concrete encasement jackets, and metalized zinc and aluminum with a protective topcoat.
8. When roadway embankment is more than 10 feet in depth, holes should be pre-augured for all piles except H-piles.

3.2.2.4 Piles for integral abutment bridges shall be designed in accordance with the methods outlined in Subsection 3.10.11.

3.2.3 Drilled Shafts

3.2.3.1 Drilled shafts shall be considered where cost and constructability may be favorable compared to spread footing or pile supported foundations. Anticipated advantages are the reduction of the quantities and cost of excavation, dewatering, and sheeting. Additionally, the use of drilled shafts may be beneficial in working within critical horizontal restrictions, or in limiting the environmental impact.

3.2.3.2 Design. Drilled shafts shall be designed in accordance with the requirements of the *AASHTO LRFD* and the following:

1. The Designer shall consider the intended method of construction (temporary or permanent casing, slurry drilling, etc.) and the resulting impact on the stiffness and resistance of the shaft.
2. In the case of single column piers, the pier column can be designed and detailed as an integral extension of the drilled shaft. If the design assumes a constant diameter of the shaft and column throughout, it is imperative that either the shaft be constructed as designed or else the design shall evaluate alternate construction details where the shaft diameter varies along its length. In addition, since the subsurface and site conditions may cause the shaft to deviate from its specified location and plumbness, the design should also establish acceptable drilled shaft construction tolerances for these deviations to allow for the pier column to be constructed in the correct location. Due to these deviations, past projects with multi column piers have had difficulty aligning the shafts with relation to each other so that the pier columns could be built in a straight line. As a result, for multi columns bents, the pier columns shall not be designed as integral extensions of the drilled shaft. A pile cap shall be provided over the drilled shafts from which the multi column pier will be built. This pile cap shall be designed to accommodate the load effects caused by the anticipated deviation of the drilled shafts from their theoretical location as allowed by the construction tolerances.
3. The lateral resistance and lateral load–deflection behavior of the drilled shaft shall be determined using soil-pile interaction computer solutions or other acceptable methods.
4. When a drilled shaft is constructed with a permanent casing, the skin friction along the permanently cased portion of the shaft should be neglected.
5. Continuous steel reinforcing shall be maintained whenever possible throughout the length of the shaft, except as noted in Paragraph 3.2.3.4. Splices should be avoided in the longitudinal steel where practical. If splices in the adjacent longitudinal reinforcement are necessary, they shall be made with mechanical reinforcing bar splicers and shall be staggered a minimum of 2'-0". Splices in the spiral confinement reinforcement shall, where necessary, be made with mechanical reinforcing bar splicers as well. The cover and detailing requirements specified in Chapter 3 of Part II of this Bridge Manual shall be satisfied. Typically, uncoated bars are acceptable in drilled shafts; however drilled shafts in harsh environments, such as marine installations, shall use coated bars.
6. The minimum clearance between reinforcing bars shall be 1⅞" and is equal to 5 times the maximum coarse aggregate size (⅜") for both, the longitudinal bars as well as the spiral confinement reinforcement, to allow for better concrete consolidation during placement. Concrete mix design and workability shall be consistent for tremie or pump placement. In particular, the concrete slump should be 8 inches ± 1 inch for tremie or slurry construction and 7 inches ± 1 inch for all other conditions.
7. The bar size and the maximum spacing (pitch) of the spirals shall meet the applicable requirements of the *AASHTO LRFD.*

The gross area of concrete (A_g) value shall assume that only 3" of concrete clear cover is provided over the spiral instead of the actual detailed, typically 5" min., clear cover. The additional cover is not needed for structural confinement of the shaft core and is only provided to improve concrete flow during concrete placement.

Based on the above, Table 3.2.3-1 below provides all applicable Spiral Bar Size/Maximum Spiral Pitch combinations that satisfy the combined requirements for the maximum bar size (#6), the minimum clearance between bars (1⅞"), and the maximum coarse aggregate used (⅜").

For larger size drilled shafts, where different Spiral Bar Size/Maximum Spiral Pitch combinations may be required, the design of the reinforcing cage shall be submitted to the State Bridge Engineer for review and approval.

3.2.3.3 Special design and detailing is required where the drilled shaft is an extension of a pier column. In these situations, column longitudinal reinforcement shall be extended into drilled shafts in a staggered manner to avoid a weakened section with a sudden change in stiffness.

3.2.3.4 As per the *AASHTO LRFD,* all designed reinforcing steel shall extend a minimum of 10 feet below the theoretical point of fixity. From this elevation, 10' below fixity, only one half of the longitudinal bars may need to be extended to the bottom of the shaft (cut every other bar), as well as a minimum of #4@12" o.c. ties only may need to be provided in this region.

The Contractor however, shall verify these minimum requirements to ensure the stability of the entire reinforcing cage is consistent with the intended concrete placement sequence and provide additional steel, if required. The required verification calculations shall be submitted by the Contractor to the Designer for approval.

3.2.3.5 For drilled shafts of bridges classified as SDC B, C, and D, the seismic detailing requirements for the plastic hinge region of the *Guide Specifications for LRFD Seismic Bridge Design* shall be satisfied.

Table 3.2.3-1: Spiral Bar Size/Maximum Spiral Pitch Combinations

Drilled Shaft Diameter, D (ft)	Spiral Bar Size	Maximum Spiral Pitch, s_{max} (in.)
3.0	# 5	3.1
	# 6	4.4
3.5	# 5	3.1
	# 6	4.5
4.0	# 5	3.2
	# 6	4.5
4.5	# 5	3.2
	# 6	4.6
5.0	# 5	3.1
	# 6	4.4
5.5	# 5	2.8
	# 6	3.9
6.0	# 5	2.5
	# 6	3.5
6.5	# 6	3.2
7.0	# 6	3.0
7.5	# 6	2.8

3.2.4 Micropiles

3.2.4.1 Micropiles shall be considered where other types of foundation support have been evaluated and cannot be efficiently used due to site constraints, existing structures or utilities. Micropiles are suitable for all soil and rock conditions except as defined herein. Micropiles shall be considered when foundations are required to be drilled through existing footings; in low overhead or restrictive (tight) site constraints; where very dense, very hard or subsurface soil conditions with boulders or other obstructions, such as utilities, are anticipated; and their use is recommended in the Geotechnical Report. Bridges and wingwalls shall be supported by permanently cased micropiles with the casing preferably terminating in a bond zone in bedrock. Termination with a bond zone in glacial till, or weathered rock that can carry the design loads if bedrock is deep, is permitted. Permanent micropiles shall not be used if the cap will be elevated in the air or water above the ground surface or mudline.

3.2.4.2 Design. Micropiles shall be designed in accordance with the provisions of *AASHTO LRFD*, current *FHWA Micropile Design and Construction Manual*, and the following:

1. The nominal geotechnical resistance to axial loads should be derived in the bond zone below the permanent casing in bedrock or other suitable material as defined in Paragraph 3.2.4.1, and any resistance from the soil and or rock above this point shall be neglected. The tip resistance of the micro pile is relatively small and shall be neglected. The Designer shall specify on the Construction Drawings whether the bond zone shall be in a rock socket or if it will be in in-situ soil.
2. The bond zone length shall be adequate to develop the nominal geotechnical resistance required by all limit states as well as the minimum penetration required for lateral stability, uplift, downdrag, scour, settlement, etc.
3. The estimation of grout to ground resistance in the bedrock bond zone shall assume the grout is placed under gravity head only which AASHTO defines as a Type A micropile. For bond zones that terminate in weathered bedrock or glacial till a Type B micropile should be used.
4. Permanent steel casing used as reinforcement shall be new Prime steel meeting the requirements of any API 5L PSL1 pipe with a yield strength of 52 ksi with SR15 supplemental requirements. The common micropile casing sizes that are typically used on MassDOT projects are given in Table 3.2.4-1 below. Other casing sizes and yield strengths higher than 52 ksi may be used provided that the Designer verifies their availability and receives prior approval by the State Bridge Engineer.
5. Permanent steel casing shall incorporate an additional 1/16” thickness for sacrificial steel corrosion protection or more if based upon soil conditions and laboratory testing.
6. Steel casing shall be drilled a minimum of 12” into intact bedrock to prevent subsidence of over burden into the uncased and/or bond zone portion of the drill hole (i.e. the rock socket). If the bond zone for the micropile is within in-situ soil, the Designer shall require that a fully cased hole be provided prior to grouting the micropiles to prevent subsidence of overburden into the bond zone.
7. A threaded central bar shall be used over the entire length of the micropile. The bars shall conform to either AASHTO M31 Grade 60 or Grade 75, or AASHTO M275 Grade 150. Approved mechanical bar couplers, if needed, may be used. Minimum grout cover should be defined on the plans for both over the bar and over the bar coupler.
8. A minimum grout compressive strength of 4000 psi shall be used unless project specific requirements require higher values.

9. Nominal resistance of micropiles to lateral loads and the load deflection behavior shall be modeled using soil pile interaction based on both structural properties and subsurface conditions at the project site.
10. No threaded casing joints shall be located within 3 feet of the pile cap. In addition, since the threaded joint results in a 60% reduction of the moment capacity of the casing, the Designer shall use the design moment diagram of the micropile to determine if threaded joints need to be prohibited to a deeper depth. This total depth of micropile where threaded joints are prohibited shall be clearly specify on the Construction Drawings.
11. The maximum batter for micropiles shall be 1:3.
12. Battered micropiles should be avoided where negative side resistance (downdrag) loads are expected or where the potential for large ground settlement around the battered micropiles is a possibility.

13. Micropile embedment into the cap shall be a minimum of 12”, with minimum edge distance and spacing as defined in the *AASHTO LRFD*. The design of the micropile cap and its connection to the pile shall be similar to other conventional piles.

Table 3.2.4-1: Common Micropile Casing Sizes Used on MassDOT Projects

Casing Outside Diameter (in.)	Wall Thickness (in.)	Cross Sectional Area (in.^{2})	Moment of Inertia (in.^{4})	Section Modulus (in.^{3})
6.625	0.432	8.40	40.49	12.22
9.625	0.472	13.57	142.51	29.61
9.625	0.545	15.55	160.80	33.41
10.75	0.500	16.10	211.95	39.43
10.75	0.545	17.47	228.10	42.44
10.75	0.595	18.98	245.53	45.68

3.2.4.3 Construction Tolerances. The following are the maximum tolerances specified for micropile construction by the MassDOT micropile special provisions. If the actual tolerances for a particular project need to be lower due to structural considerations, the Designer shall state them on the Construction Drawings.

1. Centerline of piling shall not be more than 3 inches from indicated plan location.
2. Pile shall be plumb within 2 percent of total-length design plan alignment.
3. Battered piles inclined up to 1:6 shall be within 4% of design plan alignment.
4. Battered piles inclined greater than 1:6 shall be within 7% of design plan alignment.
5. Top elevation of pile shall be plus 1 inch or minus 2 inch maximum from vertical design elevation indicated.
6. Centerline of reinforcing steel shall not be more than 3/4 inches from the indicated center of the pile.

7. Minimum volume of grout placed shall be the 110% of the theoretical volume of the whole micropile length from bottom to top at time of grouting.

3.2.4.4 Micropile Load Testing. Micropiles typically have higher design loadings in relation to their cross-sectional area than other deep foundation types. Therefore, both verification and proof load testing shall be required and specified in the contract documents.

1. Verification testing is conducted on a sacrificial micropile that is not used in the final structure. The number of verification tests to be conducted shall be based upon the geologic stratification of the site to which the test results are to be applied. If all micropiles are to have a bond zone in geologically similar bedrock, one sacrificial verification test is adequate. If the micropiles are to have a bond zone in more than one geologic strata, such as glacial till, clay, or bedrock, or geologically dis-similar bedrock, one sacrificial verification load test shall be conducted per geologic strata. The location of the verification test shall be within 10 feet of the footprint of a substructure unit, but at least 5 feet from any production micropiles. The Designer shall specify on the Construction Drawings the number and location of the verification tests that will be required.

   If the verification testing requires a change to the length of the bond zone defined on the Construction Drawings, this change shall be made by the Contractor as outlined in the micropile special provisions, approved by the Engineer, and be issued as a revision. Regardless of the verification test results, the number or the diameter of the micropiles shall not be decreased since these changes could result in a re-design of the pile cap or a change to the response of the structure.
2. Proof load testing is conducted on actual production micropiles. The number of proof load tests required by the MassDOT micropile special provisions are one micropile per substructure unit or five percent of the total number of micropiles, whichever is greater.

3.2.5 Permanent and Temporary Support of Excavation

3.2.5.1 All permanent support of excavation that is to be left in place shall preferably be steel sheeting wherever feasible. It shall be fully designed and be designated as permanent sheeting on the Construction Drawings. A unit price item shall be provided for steel sheeting in the estimate. The Designer shall verify the availability of the steel sheeting sections specified. The design shall include the following:

1. Plan view indicating horizontal limits of sheeting.
2. Cross-section indicating vertical limits of sheeting.
3. Minimum section modulus and minimum nominal yield strength of steel used.
4. Where a braced sheeting design is indicated, the design of the bracing and wales shall also be provided and shown with full dimensions on the Construction Drawings.

3.2.5.2 The Designer, in designing the permanent steel sheeting, shall assume that the bottom of excavation may be lowered by 2 feet. This lowering may be due to over-excavation or removal of unsuitable materials.

3.2.5.3 All permanent support of excavation required for the support of railroads shall preferably be steel sheeting and shall be designed by the Designer.

3.2.5.4 All sheeting that is used in conjunction with a tremie seal cofferdam shall be left in place. The Contractor shall design both the tremie seal and the cofferdam. The Designer shall indicate the estimated depth and thickness of the tremie seal, and the estimated horizontal and vertical limits of the steel sheeting for the cofferdam consistent with their design intent on the Construction Drawings. This information shall be provided for bidding and environmental permitting purposes.

3.2.5.5 For the design of the sheeting that is used as a cofferdam or as control of water, the Designer shall provide the MassDOT Hydraulics Unit with the estimated size of the cofferdam or the extent of channel that the sheeting will restrict as control of water, and the estimated duration of time that it will be in place. The MassDOT Hydraulics Unit will perform a hydraulic analysis in accordance with Section 1.6 of Part I of this Bridge Manual using the design flood return period specified in Subsection 2.6.4 (E) for temporary construction related structures and taking into account the reduction in the waterway cross section created by this temporary structure. The MassDOT Hydraulics Unit will issue a Temporary Water Control Measures Memorandum in accordance with Section 2.7 of Part I of this Bridge Manual that will provide the hydraulic design criteria and the elevation of the water that the temporary structure must safely withstand. In addition, the Designer shall indicate on the Construction Drawings the elevation at which the cofferdam should be flooded in the event that the water outside the cofferdam rises above the design water elevation, thereby causing excessive hydrostatic pressure.

3.2.5.6 The design of the tremie seal and cofferdam by the Contractor shall be in accordance with the special provision for Item No. 990.X – Cofferdam Structure No. XXX and submitted to the Designer for review as a Construction Procedure.

3.2.5.7 All temporary support of excavation required to complete the construction shall be shown on the Construction Drawings. The design of all temporary support of excavation shall be designed by the Contractor and be submitted to the Designer for review as a Construction Procedure.

3.2.5.8 All permanent and temporary support of excavation that protrudes into the soil that supports the bridge structure shall be left in place. Supporting soil shall be defined as all soil directly below the footing contained within a series of planes that originate at the perimeter of the bottom of the footing and project down and away from the footing at an angle of 45° from the horizontal.

3.2.5.9 Whether support of excavation is indicated on the Construction Drawings or not, the Contractor shall be informed by the Special Provisions that any part of the support system that protrudes into the supporting soil below the bridge structure, as defined by Paragraph 3.2.5.8, shall be cut off and left in place and no additional payment will be made for this part.

3.2.6 Gravel Borrow for Bridge Foundations

3.2.6.1 Gravel Borrow for Bridge Foundation (Item 151.1) shall be assumed to have a soil friction angle (Φ) of 37˚. The nominal bearing resistance shall be estimated using accepted soil mechanics theories for stratified soils in accordance with applicable provisions of the *AASHTO LRFD*.

Gravel for this item will be permitted up to a height of 20 feet under the footings and shall be compacted in accordance with the latest edition of the *MassDOT Standard Specifications for Highways and Bridges*. In special cases, this depth may be increased. A study should be made in each case to show that its use will result in a more economical structure. Its use is not authorized for river structures or for placement under water.

3.2.7 Crushed Stone for Bridge Foundations

In general, this material is used where water conditions prevent the use of **GRAVEL BORROW FOR BRIDGE FOUNDATIONS**. The pressure on the granular soil below the crushed stone will govern the Bearing Resistance of the crushed stone. De-watering the area and using **GRAVEL BORROW FOR BRIDGE FOUNDATIONS** compacted in the dry, or not de-watering and using **CRUSHED STONE FOR BRIDGE FOUNDATIONS** shall be investigated for feasibility and economy.

3.2.8 Foundations on Rock

3.2.8.1 If the top of rock is comparatively level and is located at a shallow depth from the proposed bottom of footing, then, for economy, consideration shall be given to lowering the footing so that it will be placed entirely on rock. The structural design of the footing shall assume a triangular or trapezoidal contact pressure distribution based upon factored loads. The maximum factored bearing pressure shall be compared to the factored bearing resistance to determine whether the bearing resistance is adequate.

3.2.8.2 If the bottom of footing will fall partly on rock and partly on satisfactory granular material, the Designer must ensure that the entire footing shall be founded on the same material throughout its bearing area. There are two strategies that can be employed depending on the rock profile and cost of the work. One strategy is to excavate the rock to a depth of about 18” below the bottom of footing and backfill with **GRAVEL BORROW FOR BRIDGE FOUNDATION**. The second strategy is to excavate the material above the rock and backfill with a minimum 3000 PSI concrete to the bottom of proposed footing elevation. When using this second strategy, an additional amount of the rock shall be excavated as needed so that the minimum thickness of the concrete backfill shall be 6”. If the subsurface exploration indicates that the top of rock surface is sloped, the Designer shall consider the possibility that the concrete backfill will slide on the rock under applied loads. Mitigation for sliding can include excavating additional rock as needed to provide stepped level bearing areas or providing dowels socketed into the rock to resist the sliding force. The Designer shall fully design the strategy to be used to ensure the stability of the foundation system and shall provide all necessary details on the Construction Drawings.

3.2.8.3 The Geotechnical Report shall provide guidance on the engineering properties of weathered and/or deteriorated rock. The Designer shall use these properties to determine the feasibility of leaving the weathered and/or deteriorated rock in place as a foundation material or removing it and replacing it with either gravel borrow for bridge foundation or a minimum 3000 PSI concrete, depending on cost. The Designer should evaluate if additional borings are required or feasible to delineate the limits of rock.

3.2.9 Pre-loaded Areas

3.2.9.1 Pre-loading or pre-loading with surcharge may be required to consolidate compressible soils and minimize long-term settlements under load. If unsuitable material is encountered, it shall be excavated prior to placing the embankment.

3.2.9.2 If the water table is higher than the bottom of excavation of unsuitable material, crushed stone shall be used in the embankment up to the proposed elevation of the bottom of footing, followed by the placement of gravel borrow for the embankment. Both these materials shall be placed during embankment construction. The amount of anticipated settlement should be accounted for in the specified top elevation of the crushed stone beneath the proposed bottom of footing. The effect of the anticipated settlement shall be considered in the design of the superstructure.

3.2.10 Scour Considerations

3.2.10.1 General. As stated in the *AASHTO LRFD*, scour is considered a change in foundation conditions, and not a force itself. Since many bridges are built on spread footing foundations, current FHWA guidance is geared primarily for avoiding the scour susceptibility of these bridges. Chapter 2 of Part I of this Bridge Manual has more information on the calculation of scour depths for various flood and streamflow conditions, where to locate the footings in response to these conditions, how to use riprap scour countermeasures, and bridges on pile foundations. In addition, Paragraph 2.3.5.9 in Chapter 2 of Part I of this Bridge Manual has guidance on locating and orienting piers and abutments for water crossings. All scour flood depth elevations are measured from the streambed Thalweg. The Thalweg is the lowest elevation point of the streambed.

3.2.10.2 Spread Footings and GRS-IBS Abutments. Spread footings without riprap countermeasures, for both abutments and piers, shall be located so that the top of the footing is set at or below the elevation of the total scour at the abutment or pier for the Scour Check Flood. See Figure 2.6.4-2 of Chapter 2 of Part I of this Bridge Manual.

3.2.10.3 Riprap Countermeasures for Spread Footings. If riprap is used to protect abutments against the effects of scour, it should be placed as shown in Figure 2.6.4-2 of Chapter 2 of Part I of this Bridge Manual and the top of the spread abutment footing should be located as shown in Figure 2.6.4-2 in Chapter 2 of Part I of this Bridge Manual. 1 For the use of countermeasures for pier protection, the Designer is referred to Hydraulic Engineering Circular No. 23 (HEC-23) *"Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance*" Volume 2_ Design Guidelines 8 - 12.

3.2.10.4 Deep Foundations. Bridges on deep foundations, such as piles, drilled shafts, and Integral Abutments, can be designed to be stable and still carry load even after a scour event and loss of streambed material. See Figure 2.6.4-3 and 2.6.4-4 for more information

3.2.10.5 Design of Deep Foundations for Scour. Regardless of whether or not riprap countermeasures are used, piles shall be designed for an un-supported pile length as measured from the bottom of the pile cap to the elevation of the total design scour or check scour at abutment as shown in Figure 2.6.4-5 of Chapter 2 of Part I of this Bridge Manual. The use of these same design criteria for deep foundations with riprap countermeasures is based on a conservative assumption that the riprap can be washed away during the maximum flood and the fact that the pile caps are set at a higher elevation than spread footings when used with riprap countermeasures.

The design requirements for the two scour flood conditions are as follows:

1. Scour Design Flood. All bridges shall be scour stable and available for use after the event under the calculated Scour Design Flood. This requires that the bridge meet all Strength and Serviceability limit states with 100% of the assumed scour present. For the Extreme Event I and II Limit State apply all load and resistance factors specified for these limit states with 50% of the calculate scour .
2. Scour Check Flood. All non-Critical/non-Essential bridges shall be scour stable at the calculated Scour Check Flood but not necessarily available for use following the Scour Check Flood event. These bridges shall be designed for the Extreme Event II Limit State with a load

factor = 0.5 for Live Load. All Critical/Essential bridges must be scour stable and available for limited use after the Scour Check Flood event. These bridges shall be designed for the Extreme Event II Limit State with a load factor of 1.0 for the HL-93 design load and dynamic load allowance.

3.2.10.6 Existing Abutments shall not be reused as scour countermeasures, unless they would be scour stable under the Scour Design Flood and Scour Check Flood events.

3.2.10.7 For bridge rehabilitation or superstructure replacement projects, or projects where the substructure units are to be retained as part of the bridge, scour countermeasures may be used to address the scour stability if the existing substructures do not fully meet all the design criteria above without them.

3.3 SUBSTRUCTURE DESIGN

3.3.1 General

3.3.1.1 Footings shall be proportioned in accordance with the standard details shown in Part II of this Bridge Manual and shall be designed for factored loads in accordance with the *AASHTO LRFD.* The passive resistance of the earth in front of a wall shall be neglected in determining local wall stability (overturning, sliding and bearing pressures). The stability of the wall during all stages of construction shall be investigated. Reinforced concrete keyways tied into footings shall preferably not be used to aid in the resistance to sliding due to the more complex construction sequence necessary to properly construct the key without disturbing the bearing soil for the rest of the footing.

3.3.1.2 Factored bearing pressures under the footings shall be calculated in accordance with the *AASHTO LRFD.* The weight of the earth in front of a wall shall be considered in computing soil pressure.

3.3.1.3 Approach Slabs. When approach slabs are used as detailed in Parts II and III of this Bridge Manual, the AASHTO's live load surcharge load on the abutment can be ignored.

3.3.1.4 In addition to the forces specified in the *AASHTO LRFD,* the non-seismic longitudinal forces for abutment design shall include the horizontal shear force developed by the bearings through either shear deformation (elastomeric bearings) or friction (sliding bearings).

3.3.1.5 Piers and abutments of a bridge over salt water will normally be protected with granite within the tidal range. The granite blocks shall be secured to the pier or abutment concrete with anchors set both into each stone and cast into the concrete. The mortar joins shall be the caulked with polysulfide caulking. Piers and abutments over fresh water do not require this protection unless the normal flow of water and seasonal water level variations are anticipated to be large.

3.3.1.6 At a minimum, the reinforcing bars used in the following elements of the substructure require protection and, so, shall be epoxy coated: backwalls, beam seats, pier caps, highway guardrail transitions, both the base and top, and the safety curb, sidewalk, and concrete barrier portions of U-wingwalls. Also, when faces of abutments, piers, wingwalls, and retaining walls are within 30 feet of a traveled way, the reinforcing bars adjacent to those faces shall be epoxy coated. If all of the reinforcing bars in the given concrete pour are to be coated, and the coated bars will never come into contact with or are to be tied to non-coated bars, then galvanized bars may be used instead of epoxy coated bars. In these situations, the Construction Drawings shall designate these bars as COATED BARS, without specifying the coating type.

3.3.1.7 Pier Protection for Bridges over Roadways. In 2018, NCHRP Report 892, *Guidelines for Shielding Bridge Piers*, was published, which was intended to update the pier protection requirements found in the *AASHTO LRFD* Article 3.6.5.1. The report identifies two specific cases when some form of pier protection is needed: 1) to protect the pier itself from impacts by heavy vehicles that can compromise the pier's structural integrity; or 2) to protect the occupants of vehicles, primarily passenger vehicles, that may impact the pier.

The report also notes that the peak load from the heavy vehicle impact that inflicts the most damage comes from the engine of the tractor. Finally, the report presents a risk-based approach for determining the need for shielding the pier for structural protection based on the probability of it experiencing such an impact. This approach includes consideration of site-specific factors, such as roadway curvature, grade, speed limit, etc., and would replace the procedure found in the *AASHTO LRFD* Commentary C3.6.5.1. The Bridge Manual pier protection requirements that follow are based on NCHRP Report 892.

1. Determine the Need for Structural Pier Protection. The Designer shall use the procedure given in NCHRP Report 892 Section 3.3 to calculate AFBC, the Annual Frequency of Bridge Collapse for an Unshielded Pier System. The actual AADT for the road being investigated shall be used in the analysis. If the AFBC is less than 0.0001 for Critical/Essential bridges or 0.001 for all other bridges, the piers do not require shielding for structural protection but should be evaluated for shielding for occupant protection. If the AFBC is greater than these values, then the Designer shall provide structural pier protection as outlined below.
2. Structural Pier Protection: for both new construction and existing piers.
   a. If, in a multi-column reinforced concrete pier, the diameter of a round pier column or the least dimension of a rectangular pier column is greater than or equal to 36", a 42" high TL-5 barrier shall be used to shield the pier. It shall be placed preferably in line with approach guardrail however, if there is insufficient offset between the edge of roadway and the pier, the barrier may be placed up against the face of the pier columns. This is permitted because the 42" high barrier will take the peak impact load from the engine block and any intrusion of a truck component behind the barrier will either be the cab or the trailer box, which does not have the same impact severity as the engine block and can be resisted without significant damage by the pier column due to its size.
   b. If, in a multi-column reinforced concrete pier, the diameter of a round pier column or the least dimension of a rectangular pier column is less than 36", a 42" high TL-5 barrier shall be placed so that the top edge of the traffic face of the barrier is 39" or more from the traffic face of the pier component being protected. If there is not enough room to accommodate this 39" offset, then a 54" high TL-5 barrier shall be placed so that the top edge of the traffic face of the barrier is preferably no closer than 20" from the traffic face of the pier component being protected. However, the 54" high barrier may be placed up against the face of the pier columns if there is no other practical option.
   c. For barriers placed in accordance with requirements 2a or 2b, the barrier shall preferably extend a minimum of 60 feet upstream of the pier to prevent a truck from penetrating behind the guardrail and still impacting the pier. For a median, if the approach barrier is narrower than the width of the pier, the barrier shall flare to meet the barrier in front of the pier.

3. Shielding for Occupant Protection: for both new construction and existing piers. For multi-column reinforced concrete piers, which do not require structural pier protection but are located within the clear zone as defined by the AASHTO Roadside Design Guide, occupant protection shielding shall be provided. This protection shall consist of the standard MassDOT W-beam guardrail found in the MassDOT Construction Standards as long as the offset distance from the face of the guardrail to the traffic face of the pier being protected is not less than the offset distance to a fixed (non-breakaway) object specified for the guardrail. If the offset distance is less than this, then a solid 42" high TL-5 barrier shall be provided. It shall be placed preferably in line with the approach guardrail, however, if there is minimal room between the edge of roadway and pier, the barrier may be placed up against the face of the pier columns. The location and length of this protection shall be designed in accordance with Chapter 5 of the AASHTO Roadside Design Guide.
4. For substructures consisting of non-redundant reinforced concrete pier columns, such as single column hammerhead piers or individual columns supporting steel cross girders, structural pier protection shall be provided in accordance with requirements 2a and 2c above if these substructures are located within the clear zone as defined by the *AASHTO Roadside Design Guide.* While these members may be designed for the 600-kip load specified in the *AASHTO LRFD* Article 3.6.5, providing structural pier protection ensures a redundant level of protection for these substructures.
5. For substructures other than multi-column reinforced concrete piers, such as pile bent substructures or steel frames, for both new construction and existing piers, if they are located within the clear zone as defined by the *AASHTO Roadside Design Guide*, structural pier protection shall be provided in accordance with requirements 2b and 2c above.
6. If occupant protection shielding is being provided for a solid wall pier, it is still preferred to install a MassDOT Construction Standards barrier in front of the solid wall. This is in consideration of the fact that the crash testing of tall wall barriers has demonstrated the potential for the head of a vehicle occupant to hit the wall during impact, resulting in serious injury. Providing an offset that is the thickness of the top of the barrier is intended to mitigate this possibility.
7. When designing a new pier consisting of reinforced concrete elements and if, based on the procedure outlined in requirement 1 above, the pier requires structural pier protection, the Designer has the option of designing the pier for the 600 kip impact load as specified in *AASHTO LRFD* Article 3.6.5 in lieu of providing structural pier protection. In this case, the pier shall be investigated for the need to provide occupant protection shielding in accordance with requirement 3 above. Also, in the case of non-redundant reinforced concrete pier columns, requirement 4 above will still apply.

3.3.1.8 Structural Protection for Abutments. Full thickness reinforced concrete abutments, either cast in place or assembled from prestressed concrete bridge elements, do not require structural protection. However, if they are located within the clear zone as defined by the *AASHTO Roadside Design Guide*, occupant protection shall be provided in accordance with Paragraph 3.3.1.7, requirements 3 and 6.

For MSE and other wall types which function as abutments by directly supporting a spread footing of a bridge stub abutment and they are located within the clear zone as defined by the *AASHTO Roadside Design Guide*, structural protection shall be provided in accordance with Paragraph 3.3.1.7 requirements 2b and 2c, so that a vehicular impact does not fail the panel, thereby compromising the backfill and consequently the bridge structure that relies on it for support. If the MSE or other wall type only retains the embankment soil and the bridge abutment has a separate foundation that does not rely on the MSE or other wall type for support and these walls are located within the clear zone as defined by the *AASHTO Roadside Design Guide*, then only occupant protection shall be provided in accordance with Paragraph 3.3.1.7 requirements 3 and 6.

3.3.1.9 Pier Protection for Bridges over Railroads. A crash wall shall be provided in accordance with the latest AREMA code or in accordance with the standards of the railroad company the bridge is over, if they are more stringent than AREMA. These crash walls shall be designed to either the *AASHTO LRFD* Article 3.6.5 collision load (600 kip), the loads specified in AREMA, or loads specified by the railroad company the bridge spans over, whichever is greater.

3.3.2 Walls: Abutments, Wingwalls, and Retaining Walls

3.3.2.1 Gravity walls. Walls of this type may be used where low walls are required, generally up to 14’ in height. When the wall is founded on sound rock the footing is omitted. The top of rock shall be roughened as necessary to provide resistance against sliding. A shear key may be provided, if necessary.

3.3.2.2 Cantilever walls. These are also the same as Semi-Gravity Walls as defined in *AASHTO LRFD*. Generally, this wall type is used in the intermediate height range (14’ to 30’) applications between gravity and counterfort walls. In those situations where a wall starts in the height range prescribed for cantilevered walls but tapers down into the height range prescribed for gravity walls, the cantilevered wall type will be used throughout instead of changing to a gravity type in mid-wall. Wall segments of variable height shall be designed, stem plus footing, using a wall height equal to the low-end wall height plus 75% of the difference in height between the low end and high end.

For spread footings, the design of the reinforcement in the toe of the footing shall not use the weight of the soil above the toe to offset the force of the upward soil pressure. Also, for spread footings, the top reinforcement in the heel of the footing shall be designed to carry the dead load of all materials above the heel, including the dead load of the heel. Live loads shall not be included. Load and Resistance Factors shall be equal to 1.0. The effect of the upward soil pressure or pile reaction will not be used to offset this design load.

Strut and tie modeling shall not be used to design reinforcement in spread footings on soil or bedrock supporting walls and abutments.

3.3.2.3 Counterfort walls. A counterfort wall design shall be considered for retaining structures and abutments higher than 30 feet. However, the economics and constructability of a counterfort wall versus a similar height cantilevered wall with a thicker stem shall be investigated.

If a railing/barrier is mounted on top of a counterfort retaining wall, the top of the wall should be detailed as a longitudinal beam that spans from counterfort to counterfort and is rigidly attached to the counterfort. The railing/barrier should be mounted on top of this beam and the beam should be designed for all of the impact loads and load effects (moment, shear, torsion) that the railing/barrier will impart as given for the Test Level of the railing/barrier in Section 13 of the *AASHTO LRFD*. The design of this beam should assume that it is unsupported between counterforts, and therefore any contribution from the wall panel should be neglected. The stability of the wall and the design of the counterfort reinforcement shall be checked in accordance with Paragraph 3.3.2.4.

3.3.2.4 Railings/barriers mounted on top of walls. Where railings/barriers are mounted on top of U-wingwalls or retaining walls, the Designer shall check the local wall stability (overturning, sliding and bearing pressures) and the stem design for a vehicular collision load using the Extreme Event II Limit State. Since this analysis is based on statics, an equivalent pseudo-static vehicular collision load shall be used. The magnitude of the pseudo-static vehicular collision force to be applied, its length of distribution along the barrier, and height above the roadway or sidewalk at which it is to be applied shall be based on the Test Level of the railing/barrier. These design criteria are given in Table 3.3.2-1 below and come from the results as published in NCHRP Web-Only Document 326:

Table 3.3.2-1: Vehicular Collision Loads for Wingwalls and Retaining Walls

MASH Test Level (MassDOT Railing)	Force (kips)	Length of Distribution (ft)	Height of Application (inch)
TL 2 and TL 3 (CT-MTL2, CP-MTL3, and CM-MTL3)	23	4	24
TL 4 (S3-MTL4)	28	5	30
TL 5 (CF-MTL5)	80	10	34

For checking local stability, in addition to all other applicable dead and live load effects, the vehicular collision load shall be assumed to engage an entire section of wall and shall have a load factor of 1.0. A section is defined as the length of wall between expansion joints. The design horizontal earth pressure from the retained soil need not be considered ($\gamma_p = 0$) to act concurrently with this load, because the wall is considered to pull away from the backfill in the instant the collision occurs and the soil does not have the time to respond before the collision is removed.

For checking the wall stem design, in addition to all applicable dead and live load effects, apply the horizontal earth pressure with $\gamma_p = 1.0$, and the vehicular collision load. For the purpose of this analysis, the vehicular collision load shall also be assumed to engage the entire stem of a section of wall, where section is defined above. The horizontal earth pressure is used here not because it acts concurrently with the collision load but because the horizontal earth pressure has already induced a strain in the reinforcing bars. The strain from the collision load adds to this strain, which results in the total strain in the rebar, and hence the total stress. Thus, the horizontal earth pressure is used here to estimate that strain.

3.3.2.5 For the design of moment slabs for railings/barriers placed on top of MSE wall systems or other wall systems that cannot resist a vehicular collision load, the methodology outlined in NCHRP Report 663 shall be used to size the barrier and moment slab system for sliding and overturning using the vehicular collision loads found in Table 3.3.2-1. The minimum length of run of a moment slab shall be 20 feet with a maximum of 60 feet. The minimum width of the moment slab shall be based on the Test Level of the railing/barrier and shall be as specified in Table 3.3.2-2. These dimensions come from the NCHRP Web-Only Document 326. The connection of the railing/barrier to the moment slab shall be designed using the loads specified in Section 13 of the *AASHTO LRFD* and not the pseudo static loads given in Table 3.3.2-1.

Table 3.3.2-2: Minimum Width of Moment Slabs

MASH Test Level	TL 2 and TL 3	TL 4	TL 5
Minimum Width (feet)	4	4.5	7

3.3.3 Piers

3.3.3.1 Piers for most structures are typically of reinforced concrete construction. Piers for grade separation structures are typically open type bents with columns. Piers for structures over railroads can be either a solid stem type or an open type bent with a crash wall conforming to AREMA requirements for pier protection, depending on an economic analysis. Piers for structures over water are typically a solid stem type. Piers for trestle type structures are typically pile bents.

3.3.3.2 For open type bents, the guidelines for establishing their geometry are presented in Chapter 5 of Part II of this Bridge Manual.

3.3.3.3 The columns shall be assumed as fully fixed at the footing, and the pier shall be designed as a rigid frame above the footing. Continuous footings founded on granular material or on piles shall be designed as continuous beams. Individual footings shall be used on ledge.

3.3.3.4 The uncracked section properties shall be used for the analyses (determination of the design load effect) of non-seismic loadings for columns, while the design of the section should be conducted assuming cracked or uncracked section, based on and consistent with, the anticipated behavior. Reduced stiffness of the section should be used for the analysis of the effects of slenderness and deflection on the design forces, as specified by the *AASHTO LRFD*.

3.3.3.5 Live load shall be positioned on the bridge deck to produce maximum stresses in the pier. To determine the maximum live load reactions on a pier, the live load shall be as required by the *AASHTO LRFD*. The multiple presence factors and the dynamic load allowance of the *AASHTO LRFD*, shall apply. Stringer reactions resulting from dead and live loads (plus dynamic load allowance) shall be considered as concentrated loads on the pier cap.

3.3.4 Reinforced Concrete Buried Structures

3.3.4.1 General. Designs of Reinforced Concrete buried structures (culverts, frames, and arches), shall conform to the requirements of Article 12.11 of the *AASHTO LRFD*.

3.3.4.2 These structures are typically designed by the Contractor/Fabricator. It is the Designer's responsibility to independently check that the design meets all the requirements of this manual and *AASHTO LRFD.*

3.3.4.3 Concrete strength shall be per the Contractor/Fabricator's design requirements.

3.3.4.4 Criteria for Loads and Live Load Distribution. Reinforced Concrete Box Culverts shall be designed for the applicable loads as specified in the *AASHTO LRFD*. The following load combinations should be considered:

1. Maximum vertical load on the roof and maximum outward load on the walls:
   $DC_{max} + EV_{max} + EH_{min} + (LL+IM)_{max} + WA_{max}$
2. Minimum vertical load on the roof and maximum inward load on the walls:
   $DC_{min} + EV_{min} + EH_{max}$

3. Maximum vertical load on the roof and maximum inward load on the walls:
   $DC_{max} + EV_{max} + EH_{max} + (LL + IM)_{max}$

For single-cell culverts, the effects of live load may be neglected where the depth of fill is more than 8 feet and exceeds the span length; for multi-cell culverts, the effects may be neglected where the depth of fill exceeds the distance between faces of end sidewalls. For both single-cell and multi-cell culverts with a skew angle of 15˚ or greater, live loads shall be applied for all depths and shall not be cut off at any preset depth.

The earth pressure shall be based on a minimum and maximum equivalent fluid pressure of 30 pcf and 60 pcf, respectively. Lateral earth pressure from weight of earth above and adjacent to a box section shall be taken as 0.5 times the vertical pressure. This value should be increased by the load factor of 1.35 for the maximum lateral earth pressure used in design. The box sections shall also be evaluated for a minimum lateral earth pressure, which may result in increased steel areas in certain locations of the culvert. The *AASHTO LRFD* allows for a 50% reduction in the lateral earth pressure in lieu of applying a minimum earth load factor of 0.9. This results in a minimum lateral earth pressure design value of 0.25 times the maximum vertical earth pressure. This minimum value is 50% of the maximum value.

In addition, the Designer shall take into consideration the potential for construction activities, such as heavy equipment movement or stockpiling of material over or adjacent to a box culvert that can induce loads in addition to the ones specified above.

3.3.4.5 Bedding and Backfill. Standard installation practices of the *AASHTO LRFD Bridge Construction Specifications* shall be followed. Side sway of the structure shall be ignored in the design of culverts provided that the fill placed around the structure shall be deposited on both sides to approximately the same elevations at the same time. No hydrostatic effect on the culvert shall be considered in design.

3.4 SEISMIC ANALYSIS AND DESIGN

3.4.1 Design Requirements

3.4.1.1 General. The goal of a seismic design can best be summarized as providing a ductile structure that will not collapse, although it may sustain significant damage. This desired performance of a bridge structure under a seismic event is primarily dependent on the ductility of the bridge elements and the provision for the dissipation of earthquake energy in a controlled manner that will not cause sudden catastrophic failure of the main load supporting elements, supplemented with, but not entirely replaced by, a static structural design based on forces and displacements of an assumed earthquake. Therefore, MassDOT stresses good detailing and the use of Earthquake Resisting Systems (ERS) even for bridges that are classified as SDC A and requires that most SDC A bridges be detailed in accordance with higher SDC requirements.

The standard MassDOT “floating bridge,” or a superstructure fully carried on elastomeric bearings without defined fixed or expansion bearings, is to some degree an ERS with the elastomeric bearings providing some measure of isolation. This standard concept relies on the keeper blocks, shear keys and backwalls to withstand the required displacement that must be accommodated in the substructure, and these components are designed elastically to do so. The standard bearing assembly as detailed in Chapter 14 of Part II of this Bridge Manual provides for the “floating bridge” concept and shall be used wherever possible, especially for new bridges. However, in cases where it is not feasible to provide keeper blocks, shear keys and backwalls as restraints (e.g. bridge preservation projects), bearings with anchor bolts may be allowed as seismic restraints to the superstructure with prior MassDOT approval. This restriction is due to the fact that anchor bolts, in reality, provide discrete restraint points and not a continuous restraint to the superstructure. For higher seismic accelerations, superstructure displacement motion may not load all anchor bolts uniformly, which may result in some anchor bolts being overstressed and potentially failing, which can contribute to a progressive failure of the other anchor bolts, leading to the loss of restraint of the bridge superstructure.

3.4.1.2 As specified in Subsection 3.1.1 of this Bridge Manual, all seismic analysis and design of bridges shall be performed in accordance with the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Except where noted, these Guide Specifications shall be used instead of the seismic provisions in the *AASHTO LRFD*. In lieu of the simplified method contained in *the AASHTO Guide Specifications for LRFD Seismic Bridge Design*, either a more refined Single-Mode Spectral Analysis or a Multi-mode Spectral Analysis, depending on the complexity of the structure, may be used to determine the seismic demand for the earthquake design of conventional, regular and historic structures.

If Designers use the more refined analysis method, they shall model the structure, including the bearings, based on the actual anticipated behavior of the structure. For most bridges falling into this category, this additional effort is not justified. However, for large structures, with long spans and large dead loads, the more reasonable seismic demands based on this more refined analysis could result in substructure construction savings. If a multi-mode spectral analysis is used to determine the seismic demand, all subsequent design shall be done in accordance with the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and in accordance with the requirements of the Paragraphs that follow.

3.4.1.3 Critical/Essential bridges in Massachusetts, as defined in Subsection 3.1.2, shall be designed for a seismic hazard corresponding to a Two Percent Probability of Exceedance in 50 years (approximately 2500-year Return Period). A site-specific hazard analysis is not automatically required for Critical/Essential bridges, except in those situations described in Paragraph 3.4.2.3, since the enhanced performance is obtained through the modified SDC C detailing to ensure ductile behavior where shown in Figures 3.4.3-2 and 3.4.3-4.

3.4.1.4 Background. The Guide Specifications differ from the procedures provided in the *AASHTO LRFD* in that they use a displacement-based design approach, instead of the traditional, force-based "R-factor" method. This new approach allows for a more accurate calculation of the actual inelastic seismic capacity of a bridge than by using the approximate inelastic ductility estimates in the R-factors. The application of this method varies from a simplified implicit displacement check procedure to a more rigorous pushover assessment of displacement capacity depending on the Seismic Design Category (SDC) that has been assigned. The SDC for each bridge is based on the Design Spectral Acceleration Coefficient at the 1.0 sec period (S_{D1}), which is the product of the site coefficient (F_v) and the spectral acceleration (S_1). Seismic Design Categories vary from SDC A through SDC D.

3.4.1.5 Seismic Design Strategy (SDS). The MassDOT standard is a Ductile Substructure with an Elastic Superstructure, or a Type 1 SDS as defined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. As noted in Paragraph 3.4.1.1, the standard MassDOT floating bridge structure, as detailed in Parts II and III of this Bridge Manual, can behave as a Type 3 isolation SDS for low seismic displacement demands that can be anticipated for bridges categorized as SDC A and allowed by the closed cell foam, even though it is not specifically designed as such. For this reason, the MassDOT floating bridge structure shall be the first choice for new construction for conventional bridges.

In order to ensure ductile behavior, all substructures regardless of SDC shall be designed using the Limited-Ductility Response method as defined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Article 4.7.1 (i.e. “ductility demand” $\mu_D \leq 4.0$). In addition, the Local Displacement Capacity check specified in *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Article 4.8.1 shall be performed. For bridges classified as SDC A and SDC B, the Designer shall use the SDC B Equation 4.8.1-1 and, for bridges classified as SDC C, the SDC C Equation 4.8.1-2 regardless of the detailing requirement called for in the Paragraphs below. The minimum 15 foot clear height limitation for a column that is being checked by these equations shall not apply to standard MassDOT multi-column piers due to the fact that the ductility demand is usually low.

If the seismic design of a bridge structure would benefit from the use of an isolation system, the Designer can take the standard MassDOT floating bridge concept and, by providing isolation elements and allowing for the anticipated displacement demand and designing the bridge elements to be an isolated Type 3 SDS with the prior approval of the State Bridge Engineer. If the isolation system provided does not have a restraining element to prevent the bridge superstructure from moving off of the substructure units, shear keys, backwalls and keeper blocks shall be provided, however the thickness of the closed cell foam lining them shall be sized to be 1.5 times the anticipated seismic displacements. Seismic isolation can be achieved through either the use of PTFE Bearings that reduce the inertial superstructure forces on the substructure, elastomeric bearings that can accommodate the anticipated seismic displacements or full isolation bearings that allow for energy dissipation. The request for using a Type 3 SDS shall include the proposed seismic isolation strategy and the methodology for accommodating the anticipated seismic displacements without engaging the substructure.

Type 2 SDS shall not be used for the design of MassDOT bridges unless the Designer can demonstrate that using this SDS will result in a structure that will have an enhanced seismic performance versus a Type 1 or Type 3 SDS and that the ductile elements in the pier cross frames shall not suffer irreparable damage during a seismic event.

3.4.2 Seismic Hazard Maps

3.4.2.1 For all non-Critical/non-Essential conventional bridges the seismic hazard maps provided in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* shall be used. These maps represent a seismic hazard corresponding to a Seven Percent Probability of Exceedance in 75 years (approximately 1000-year Return Period). The map of The Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0 second (S_1) in this series indicates that this acceleration coefficient for Massachusetts varies between approximately 2.7 and 4.1 percent of “g” for a reference Site Class B. As a result, the vast majority of bridges in Massachusetts will be classified as SDC A; however, since bridges located adjacent to the Vermont - New Hampshire border see higher accelerations, they may fall into a higher SDC depending on the soil type.

3.4.2.2 For all Critical/Essential conventional bridges the maps depicting the 2500-year return accelerations that shall be used for analysis and design are attached as an Appendix to Part I of this Bridge Manual. The three maps in this Appendix were taken from the USGS website and were edited to show just the accelerations in Massachusetts. They are analogous to the 1000-year return seismic hazard maps found in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* since they provide the same three data points that are needed to construct the Design Response Spectrum using the General Procedures outlined in Article 3.4.1 in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Once the Designer constructs the Design Response Spectrum for the 2500-year return design seismic event, all subsequent analysis and design will be done in accordance with the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

3.4.2.3 For all non-conventional bridges, a site-specific seismic hazard analysis shall be used in place of the seismic hazard maps noted in Paragraphs 3.4.2.1 and 3.4.2.2 to determine the actual accelerations at the bridge site. These accelerations shall be for either the 2500-year or 1000-year return period earthquake, depending on whether the bridge is Critical/Essential or not.

3.4.2.4 A Site-Specific Ground Motion Response Analysis may also be used for bridge rehabilitation projects if the expense of such analysis is economically justified or as required in the Guide Specifications for a bridge rehabilitation for soils that fall into Site Class F.

3.4.3 Analysis and Design Methodology

3.4.3.1 Figures 3.4.3-1 through 3.4.3-4 below depict the general analysis and design flowcharts for the seismic analysis and design of bridges in Massachusetts.

3.4.3.2 For superstructure replacement projects or bridge rehabilitation projects, the Designer shall analyze the existing substructure units using the procedures specified above as if it were a new bridge and seismic strength and detailing deficiencies shall be identified in the substructure units. The seismic detailing requirements need not be greater than what is required for the SDC classification of the bridge, except that for SDC A, seismic detailing shall be as specified in the *AASHTO LRFD* for Seismic Zone 1. For the remainder, SDC B, C and D, the detailing shall be as required in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for the given SDC.

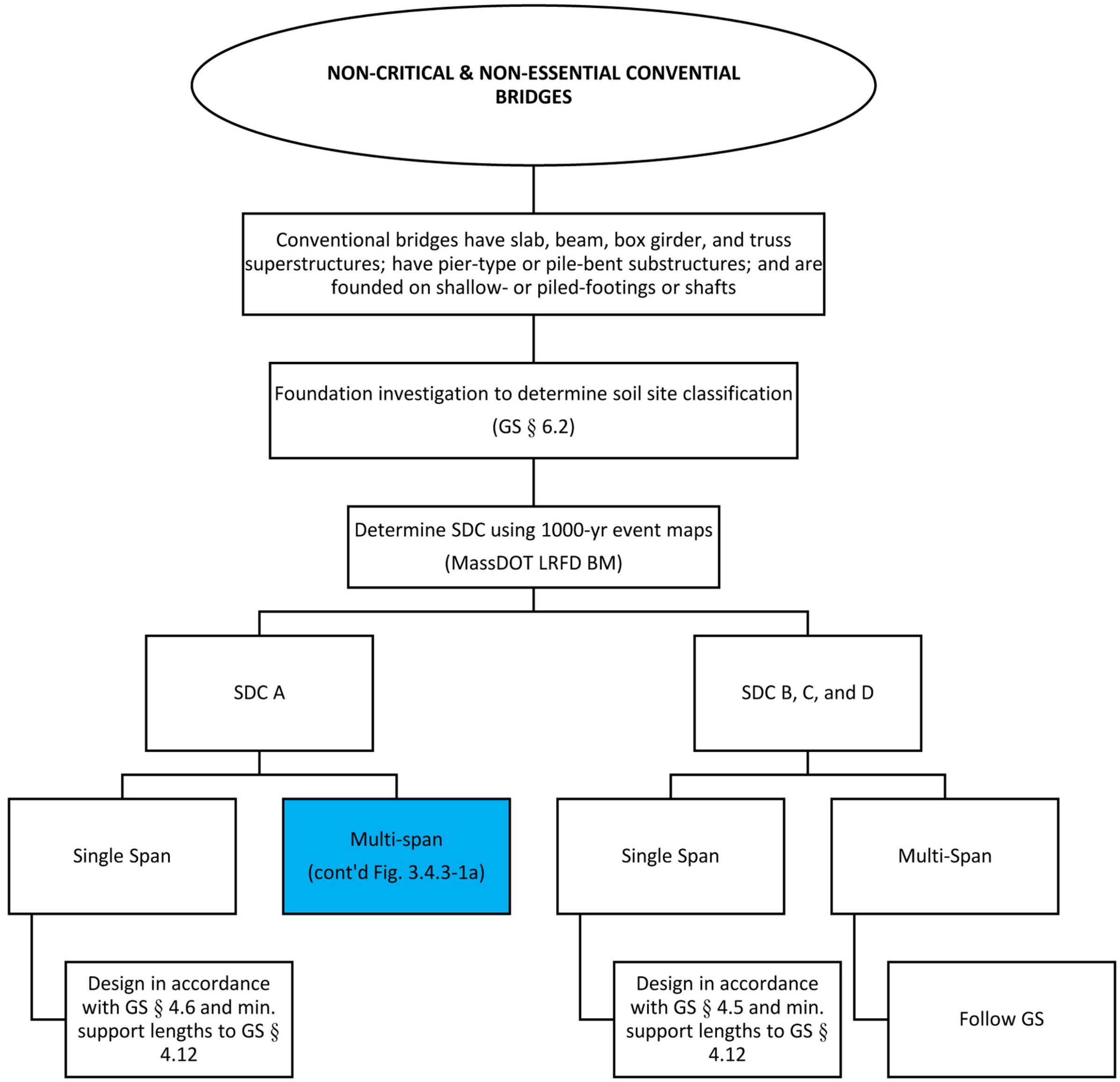
For those structures where the replacement of the existing substructure units is not reasonable, or for structures where the substructure is in otherwise reusable condition and could be upgraded to the new service design demands, i.e. HL-93, HS-25, etc., the designer shall consider the strategies contained in the *FHWA MCEER SEISMIC RETROFITTING MANUAL FOR HIGHWAY STRUCTURES: PART 1-BRIDGES* for seismic retrofit alternatives. The designer shall evaluate appropriate higher level analysis methodologies both for force distribution and for capacity/demand, such as multi-mode analysis, seismic isolation, site specific hazard analysis, non-linear static (push over) analysis, and elastic/plastic capacity (moment curvature) analysis and develop a seismic evaluation strategy. This strategy shall be presented to the MassDOT State Bridge Engineer for review and approval prior to implementation.

The intent is to confirm by analysis the structure has adequate strength and ductility to meet the seismic demands it is likely to see. If the higher level analysis demonstrates the structure is still inadequate, retrofit measures will then be evaluated for construction cost and feasibility to upgrade the structure seismically. For structures where the substructure "must" be re-used, every effort shall be made to bring the structure's capacity up to the demand.

The seismic evaluation results shall be included in the Preliminary Structure Report as part of the substructure reuse evaluation. The expense of performing the additional analysis can be justified in terms of potentially eliminating the construction cost of replacing a salvageable substructure, better targeting specific locations for upgrade within the substructure and reducing that cost or ultimately confirming that the substructure cannot be reasonably retrofit and must be replaced.

3.4.3.3 The load factor for Live Load γ_{EQ} shall be taken as 0.0. This is based upon research conducted at the University of Nevada, Reno (Center for Civil Engineering and Earthquake Research), which concluded that at low amplitude motions, with shear keys still intact, the live load on the bridge actually had a beneficial effect. However, once the shear keys failed, the performance of the structure would be closer to the no-live load case.

3.4.3.4 For the purpose of design requirements, MassDOT defines superstructure/substructure connections as those elements that transfer shear or shear and axial loads between one component and another. Generally, they include reinforced concrete shear keys, keeper blocks, backwalls, and/or anchor bolts of bearing devices, if used.



GS: AASHTO Guide Specifications for LRFD Seismic Bridge Design
LRFD: AASHTO LRFD Bridge Design Specifications

Figure 3.4.3-1: Flowchart - Seismic Design of Non-Critical & Non-Essential Bridges

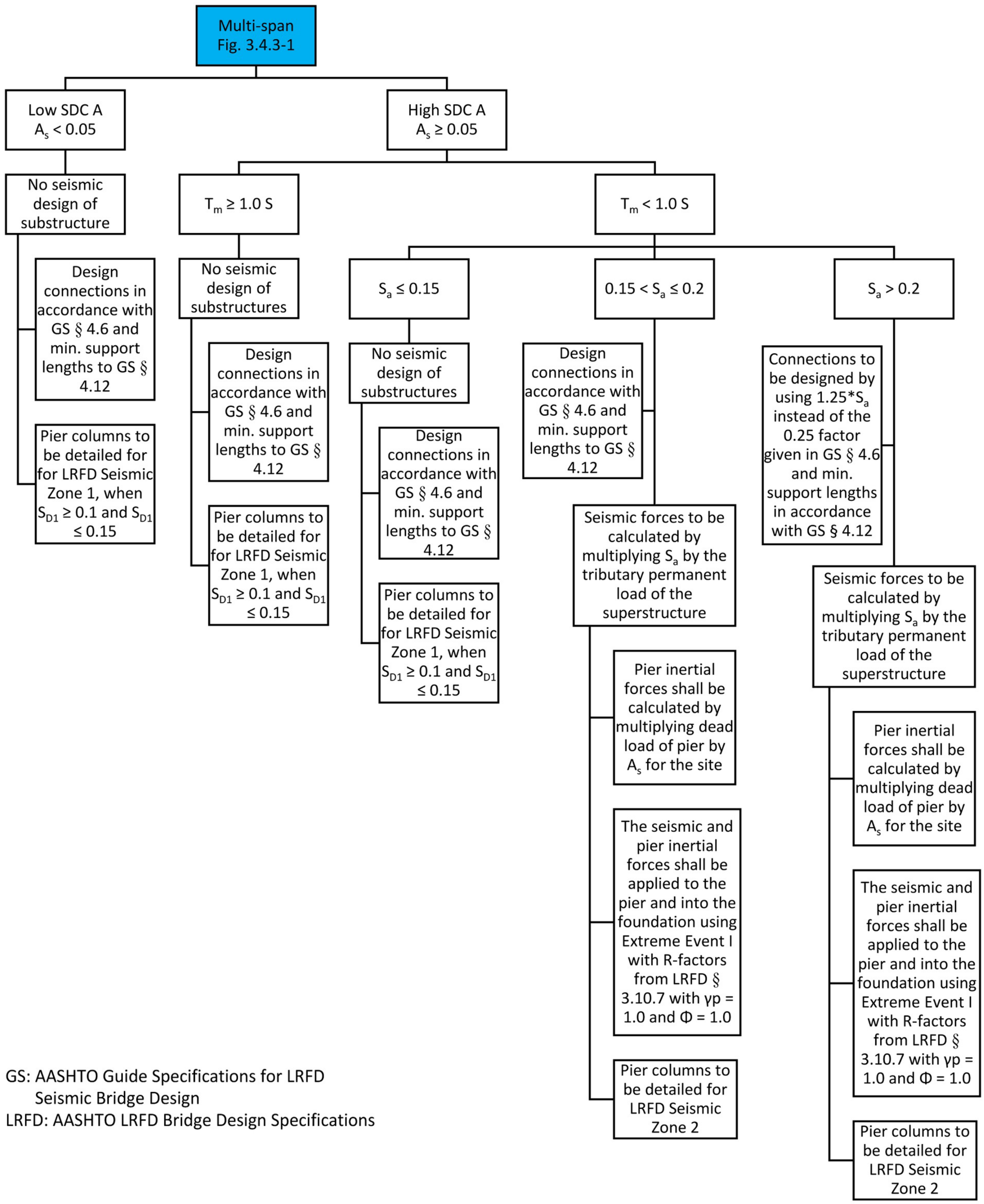
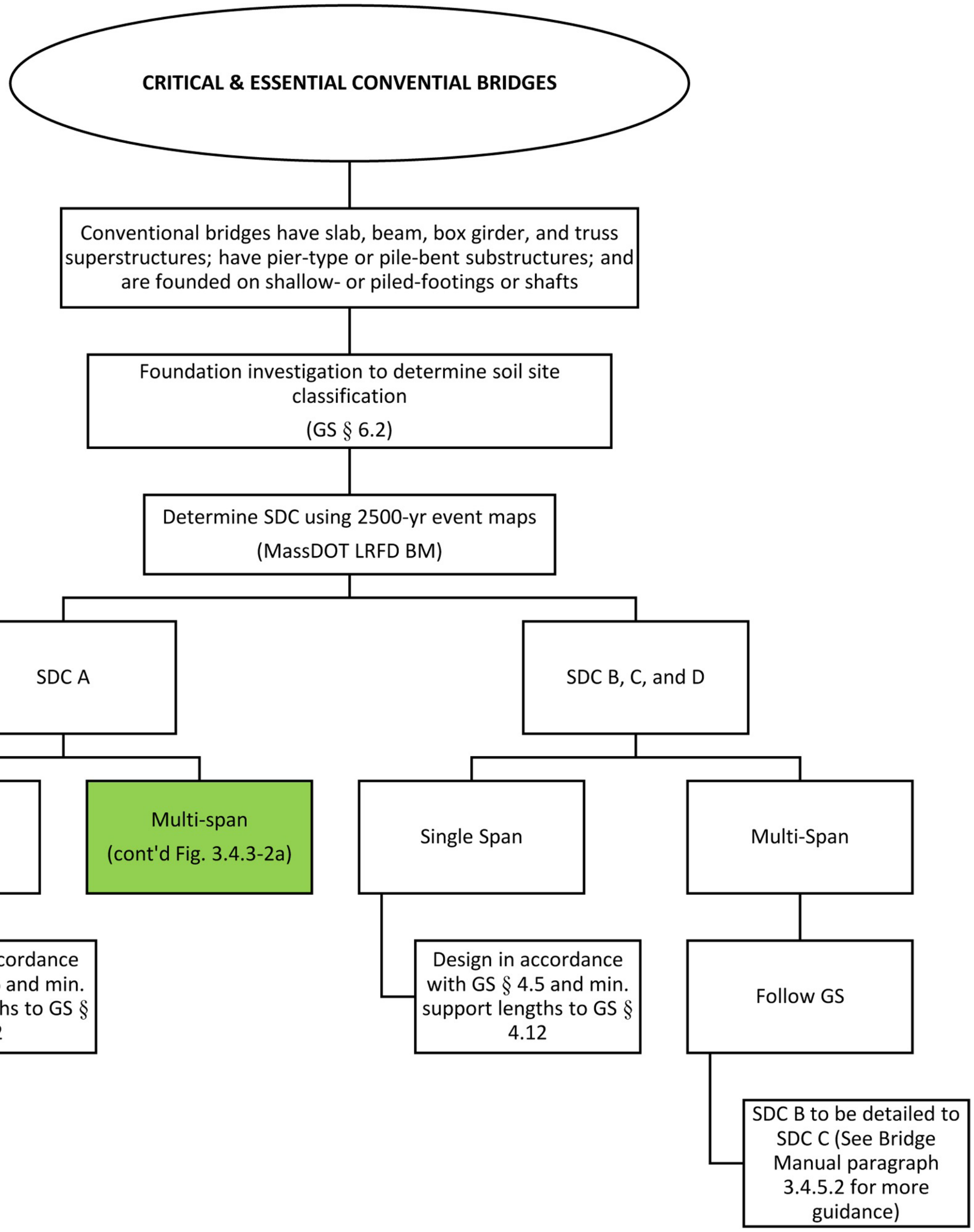


Figure 3.4.3-1a: Flowchart - SDC A Seismic Design of Multi Span Non-Critical & Non-Essential Bridges



GS: AASHTO Guide Specifications for LRFD Seismic Bridge Design
LRFD: AASHTO LRFD Bridge Design Specifications

Figure 3.4.3-2: Flowchart - Seismic Design of Critical & Essential Bridges

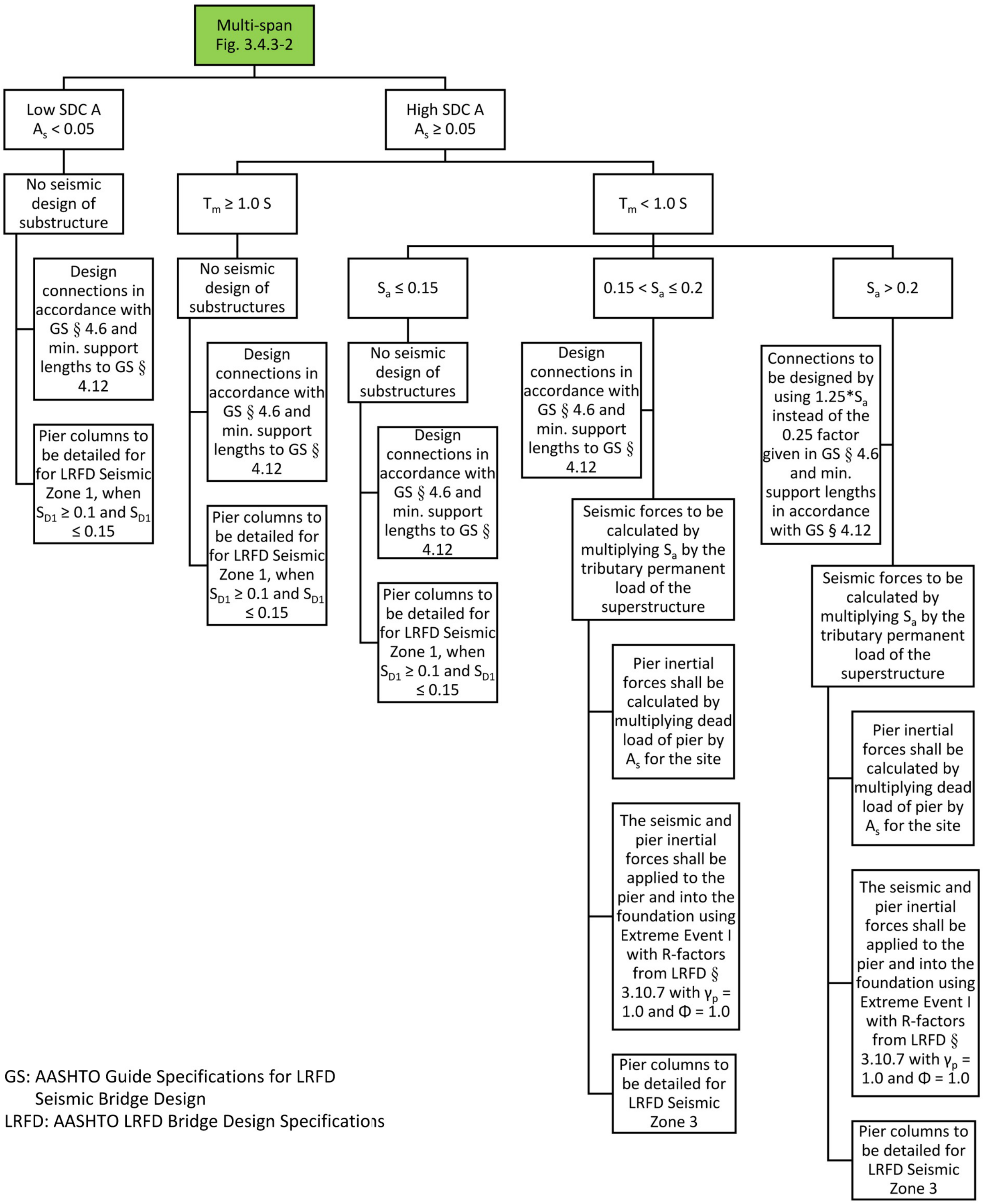


Figure 3.4.3-2a: Flowchart - SDC A Seismic Design of Multi Span Critical & Essential Bridges

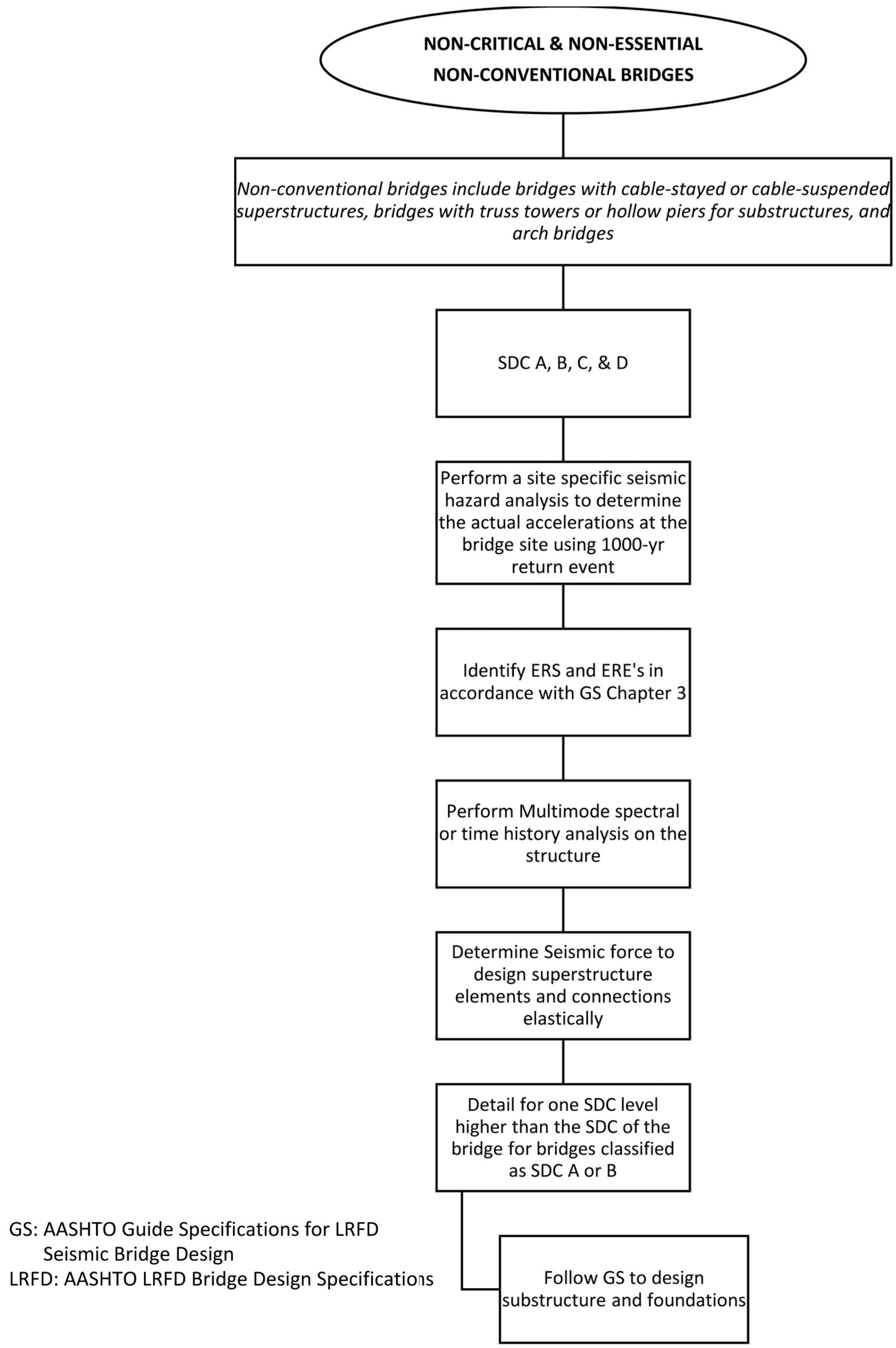


Figure 3.4.3-3: Flowchart - Seismic Design of Non-Conventional Non-Critical & Non-Essential Bridges

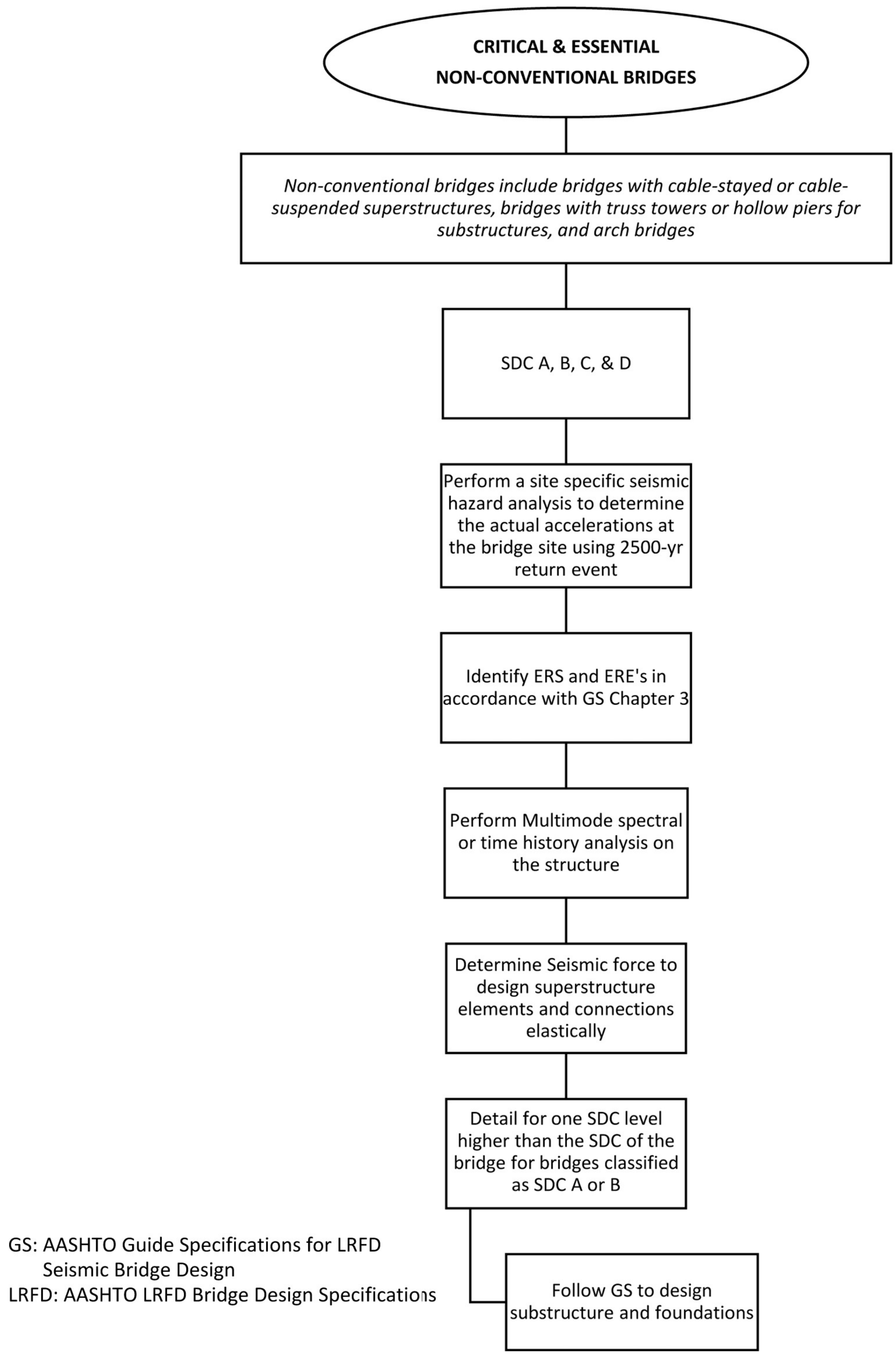


Figure 3.4.3-4: Flowchart - Seismic Design of Non-Conventional Critical & Essential Bridges

3.4.4 Design Procedures for Conventional Bridges Classified as SDC A

3.4.4.1 Introduction. Prior to 2014, both the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and the *AASHTO LRFD* required that, for bridges classified as SDC A, the horizontal seismic forces used to design the superstructure/substructure connections had to be carried down to the foundation. This requirement resulted in some substructures for SDC A bridges being designed for forces that were higher than if those bridges were in SDC B.

At its December 2013 midyear meeting, the AASHTO T-3 Technical Committee for Seismic Design reconsidered this requirement and decided that substructures and foundations for bridges classified as SDC A need not be designed for any seismic forces, including those that were specified for the design of the superstructure/substructure connections. T-3 felt that bridge substructures designed for normal Strength Limit States load effects would have enough capacity for seismic accelerations up to and including 0.15g, the defining limit of SDC A. The superstructure/substructure connections themselves would still have to be designed for the higher loads to ensure that these connections would not fail.

At this meeting, T-3 also considered that, depending on the site classification and the period of the structure, the superstructure forces for some SDC A bridges might exceed the connection design forces. Instead of developing specific design requirements, T-3 decided to put this as a caveat in the C4.6 Commentary in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The MassDOT design procedures for bridges classified as SDC A that follow are intended to align the MassDOT Bridge Manual with these latest changes in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and to provide design steps that address the possible situations the commentary cautions about.

3.4.4.2 Definitions. SDC A is as defined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* as the site where the design spectral acceleration coefficient at the 1.0 sec period (SD1) is less than 0.15.

"Low" SDC A is the site where the acceleration coefficient, A_S, is less than 0.05.

"High" SDC A is the site where the acceleration coefficient, A_S, is greater than or equal to 0.05.

3.4.4.3 For all conventional bridges, both single and multi-span, classified as SDC A, the abutments do not have to be designed for seismic forces nor do the inertial mass of the abutment itself or seismic soil forces need to be considered in design. Nevertheless, the superstructure/substructure connections shall be designed for the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and the minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. To be consistent with Paragraph 3.4.3.3, the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* shall only be based on the tributary permanent loads and shall not include the tributary live loads. These design provisions do not apply to semi-integral abutments where the superstructure end diaphragm overhangs the back of the abutment. For the design requirements for these types of abutments, see Paragraph 3.4.6.1.

3.4.4.4 For all conventional multi-span bridges located in the "Low" SDC A sites, substructures do not have to be designed for seismic forces. Nevertheless, the superstructure/substructure connections (at both abutments and piers) shall be designed for the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and the minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Detailing of the reinforcement at the top and bottom of the columns shall be as required in the *AASHTO LRFD* for Seismic Zone 1 where the response acceleration coefficient S_{D1} is greater than or equal to 0.10 and less than or equal to 0.15. For "Low" SDC A sites where the response acceleration coefficient S_{D1} is less than 0.10, detailing of the reinforcement at the top and bottom of the columns shall only be as required in Part II of this Bridge Manual.

3.4.4.5 For conventional multi-span bridges, both non-Critical/non-Essential and Critical/Essential, located in the "High" SDC A sites, the Designer shall calculate the period of the bridge, T_m, using the *Procedure 1: Equivalent Static Analysis (ESA)* methodology found in the Commentary article C5.4.2 in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

1. If the period of the bridge, T_m, is greater than or equal to 1.0 second, the substructures do not have to be designed for seismic forces. Nevertheless, the superstructure/substructure connections (at both abutments and piers) shall be designed for the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* using only the tributary permanent loads, and the minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Detailing of the reinforcement at the top and bottom of the columns shall be as required in the *AASHTO LRFD* for Seismic Zone 1 where the response acceleration coefficient S_{D1} is greater than or equal to 0.10 and less than or equal to 0.15.
2. If the period of the bridge, T_m, is less than 1.0 second, the Designer shall calculate the design response spectral acceleration coefficient, S_a, using the appropriate seismic hazards maps as specified in Paragraphs 3.4.2.1 or 3.4.2.2 depending on whether the bridge is Critical/Essential or not.
    a. If S_a is less than or equal to 0.15: the substructures do not have to be designed for seismic forces. Nevertheless, the superstructure/substructure connections (at both abutments and piers) shall be designed for the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* using only the tributary permanent loads, and the minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Detailing of the reinforcement at the top and bottom of the columns shall be as required in the *AASHTO LRFD* for Seismic Zone 1 where the response acceleration coefficient S_{D1} is greater than or equal to 0.10 and less than or equal to 0.15.
    b. If S_a is greater than 0.15 and less than or equal to 0.20: the superstructure/substructure connections (at both abutments and piers) shall be designed for the seismic forces as defined in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* using only the tributary permanent loads, and the minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The S_a shall be multiplied by the tributary permanent load of the bridge superstructure at each pier to calculate the seismic forces to be applied to the pier and into the foundation for their design, both longitudinally and transversely. The analysis shall follow the requirements of the *AASHTO LRFD* for Seismic Design using the Extreme Event I Limit State from Table 3.4.1-1 with a γ_p =1.0, γ_{eq} =0.0, and a Resistance Factor ϕ =1.0, and in the Combination of Seismic Forces Effects specified in Article 3.10.8. Since the MassDOT Seismic Design Strategy (SDS) is to use a Ductile Substructure, the R-Factors from Article 3.10.7 of the *AASHTO LRFD* shall be applied to these forces. For Critical/Essential bridges, use the Critical Operational Category R-Factors. The check of the substructure units for resistance to sliding and overturning shall also be based on the

Extreme Event I load and resistance factors. The longitudinal and confining reinforcement for the piers shall be detailed in accordance with the requirements of Seismic Zone 2 for non-Critical/non-Essential bridges and the requirements of Seismic Zone 3 for Critical/Essential bridges.

c. If the S_a is greater than 0.20: then the factor by which the tributary permanent load is multiplied to calculate the seismic force used in designing the superstructure/substructure connections, shall equal 1.25 S_a. This factor shall be used in place of the 0.25 factor given in Article 4.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The S_a shall be multiplied by the tributary permanent load of the bridge superstructure at each pier to calculate the seismic forces to be applied to the pier and into the foundation for their design, both longitudinally and transversely. The analysis shall follow the requirements of the *AASHTO LRFD* for Seismic Design using the Extreme Event I Limit State from Table 3.4.1-1 with a γ_p =1.0, γ_{eq} =0.0, and a Resistance Factor ϕ =1.0, and in the Combination of Seismic Forces Effects specified in Article 3.10.8. Since the MassDOT Seismic Design Strategy (SDS) is to use a Ductile Substructure, the R-Factors from Article 3.10.7 of the *AASHTO LRFD* shall be applied to these forces. For Critical/Essential bridges, use the Critical Operational Category R-Factors. The check of the substructure units for resistance to sliding and overturning shall also be based on the Extreme Event I load and resistance factors. The longitudinal and confining reinforcement for the piers shall be detailed in accordance with the requirements of Seismic Zone 2 for non-Critical/non-Essential bridges and the requirements of Seismic Zone 3 for Critical/Essential bridges.

3.4.4.6 When designing reinforced concrete bridge substructures for the seismic loads in accordance with Paragraph 3.4.4.5 (2b), the effective (reduced due to cracking) properties of the section may be used per Article 5.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* to reduce the seismic forces.

3.4.4.7 Pier inertial load effects for SDC A Bridges.

1. For multi-span bridges located in “Low” SDC A sites, the pier inertial forces need not be considered.
2. For multi-span bridges located in “High” SDC A sites, the pier inertial forces shall be considered by multiplying the Dead Load weight of the pier by the A_S for the site. This inertial load effect should be applied at the Center of Gravity of the pier using Extreme Event I Limit State from the *AASHTO LRFD* Table 3.4.1-1 with a γ_p =1.0 and a Resistance Factor ϕ =1.0.

3.4.4.8 For semi-integral abutments, where the backwall overhangs the back of the abutment, follow the procedures in Paragraph 3.4.6.1.

3.4.4.9 For non-conventional bridges located in the “High” SDC A sites, see Subsection 3.4.7 for additional guidance and design requirements.

3.4.5 Design Procedures for Bridges Classified as SDC B, C, or D

3.4.5.1 For single-span bridges classified as SDC B, C or D, a detailed seismic analysis to determine the design earthquake loading is not required. Nevertheless, the following minimum design and detailing requirements shall be satisfied:

- The superstructure/substructure connections shall be designed both longitudinally and transversely to resist a horizontal seismic force as specified in Article 4.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for single-span bridges.
- Procedures specified in Article 4.5 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* shall be followed to check the abutments and foundations for seismic loads.
- The minimum support lengths shall be checked in accordance with Article 4.12 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design.*

3.4.5.2 For conventional multi-span bridges classified as SDC B, C or D, a seismic analysis shall be performed in accordance with the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* using the appropriate seismic hazard maps as specified in Paragraphs 3.4.2.1 and 3.4.2.2 depending on whether the bridge is Critical/Essential or not. The forces and displacements derived from this analysis shall also be used with the appropriate SDC procedures in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* to design the substructures and foundations. In addition, for Critical/Essential bridges classified as SDC B, column longitudinal and confinement reinforcement shall be designed and detailed in accordance with the requirements of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for SDC C by applying the requirements of Articles 8.6, 8.7 as modified below, and 8.8 as modified below, except that the SDC B elastic seismic forces shall be used instead of the SDC C forces associated with the overstrength moment. The requirements of Article 8.7.2 in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* shall only apply to bridges that are actually classified as SDC C or SDC D and not to those that are actually classified as SDC B but are being detailed to a higher level for improved ductility. For checking the requirements of Article 8.8.12 and Article 8.8.13, the SDC B elastic seismic forces shall be used instead of the moments derived from 1.25 times the overstrength moment of the embedded column.

3.4.5.3 Resistance to Sliding and Overturning. For bridges classified as SDC B, the check for sliding and overturning shall be as outlined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* except that elastic seismic forces shall be used instead of the overstrength moment. For bridges classified as SDC C or D, the check for sliding and overturning shall be as outlined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

3.4.5.4 For the seismic analysis of the reinforced concrete bridge components (columns, caps, etc.) the effective (reduced due to cracking) properties of the section shall be used as per Article 5.6 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

The use of effective stiffness will generally increase the period of vibration of the structure and consequently may decrease the forces depending on the shape of the design response spectrum. However, the displacements will be increased, which may be critical in evaluating seat lengths, bearing movement capacities and P-Δ effects. Thus, although some conservatism in force level may be lost by using effective stiffness in the analysis, more realistic displacements and more accurate forces will result.

3.4.5.5 For multi-span bridges classified as SDC B, C, or D, the application of substructure inertial effects and seismic soil forces shall be in accordance with the provisions of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

3.4.6 Other Seismic Design Considerations

3.4.6.1 Semi-integral abutments classified as SDC A, B, C, or D, where the superstructure end diaphragm overhangs the back of the abutment, in addition to all design requirements for abutments in those SDC's, shall also be checked for resistance to overturning from 100% of the seismic active soil force calculated by the Mononobe-Okabe method. The accelerations (A_S) used shall be calculated for the bridge's site design response spectrum and earthquake return frequency (1000 year or 2500 year). The superstructure shall only impart vertical reaction loads and shall not provide any horizontal restraint nor shall the seismic superstructure force be used in conjunction with the seismic soil forces. If this seismic demand is greater than the calculated Strength Limit demand, the abutment reinforcing shall be re-designed for these higher forces.

The end diaphragm shall be designed for not more than 70% of the seismic passive soil pressure that can be applied to the end diaphragm.

This additional requirement for semi-integral abutments is to ensure that this type of abutment has sufficient resistance to overturning during a seismic event. Since the superstructure cannot act as a strut because there is no backwall for it to engage, this type of abutment must rely on its own stability to prevent it from tipping over and resulting in the failure of the bridge structure.

3.4.6.2 For superstructure replacement projects or bridge rehabilitation projects, when analyzing substructures founded on piles, determining the amount of ductility in the pile system so that they can withstand the increased level of displacement, will also allow a reduction of the horizontal acceleration of 50%.

In addition, for these types of projects and where piers are on spread footings, allowing them to rock on their footings is an acceptable strategy for accommodating seismic demands.

3.4.6.3 Background. Abutments on spread footings are allowed a reduction in the horizontal acceleration because recent work under NCHRP Report 611 has concluded that a permanent ground displacement associated with a horizontal acceleration of $0.5A_S$ will in most cases be less than 1 to 2 inches. This is typically the dimension that is provided the MassDOT standard details before the abutment backwall engages the end diaphragm of the superstructure. Abutments on piles are considered to be restrained from movement and so the full acceleration coefficient A_S is used since this will develop the most shear force effect on the piles and will provide a conservative design where the piles will not fail below ground.

The use of only the wall inertia and the active static soil pressure are prescribed in consideration of the recent research that indicates that seismic soil forces and inertia wall forces are out of phase in their application to the abutment. Furthermore, seismic soil forces are not used because for them to develop, shearing of the soil mass would be required. Considering the intensity of the type of earthquake that would be experienced in Massachusetts and its duration of shaking, there is low probability that the seismic soil force, as predicted by Mononobe-Okabe, would develop. The load case specified, wall inertia force with the active soil force, has a greater probability of occurring during a Massachusetts earthquake. The passive soil pressure limitation is based on the limitations for using the passive abutment resistance as a Permissible Earthquake Resisting Element.

3.4.7 Design Procedures for Non-Conventional Bridges Regardless of SDC

3.4.7.1 For all non-conventional bridges, an Earthquake Resisting System (ERS) and Earthquake Resisting Elements (ERE) shall be identified in accordance with Chapter 3 of the *AASHTO Guide*

Specifications for LRFD Seismic Bridge Design. "Permissible ERS and ERE with Owner's approval" require prior approval by the State Bridge Engineer before being used in a design.

3.4.7.2 In conjunction with the site-specific seismic hazard analysis discussed in Paragraph 3.4.2.3, a multi-mode spectral or time history analysis shall be performed on the structure as appropriate. The return period to be used shall be based on whether the bridge is non-Critical/non-Essential (1000-year return period) or Critical/Essential (2500-year return period). This analysis will provide the modal shapes, forces and displacements of the superstructure that will be used to design the individual elements of the superstructure and their connections. All connections shall be designed elastically. The forces and displacements derived from this analysis shall also be used with the appropriate SDC procedures in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* to design the substructures and foundations. All non-conventional bridges classified as SDC A or B shall be detailed for one SDC level higher than the actual SDC of the bridge. For example, a non-conventional SDC A bridge shall be designed for SDC B, and an SDC B bridges shall be designed for SDC C.

3.4.8 Modeling and Design of Bridge Bearings for Seismic Analysis

3.4.8.1 Ductile Substructure, Type 1 SDS. The distribution of forces for bridges with elastomeric expansion bearings (including elastomeric bearings that are not bonded to sole and masonry plates) should be based on the assumption that none of the bridge bearings will slide during the seismic event. This is the typical normal assumption for the distribution of seismic forces to substructure units for the standard MassDOT "floating bridge". When using the simplified method, bearings shall be assumed to be pinned at each substructure unit in each direction, longitudinal and transverse, in the analysis. Keeper blocks/shear keys/backwalls (and anchor bolts where allowed) shall be placed on substructure units consistent with the presumed restraint direction (longitudinal and/or lateral) of the superstructure used in the analysis.

If the superstructure is not restrained in the longitudinal direction at the piers with shear keys (or anchor bolts if allowed), the backwall of each abutment shall be designed to act as the longitudinal restraint for the entire bridge superstructure. The backwalls shall be designed for the full seismic force from the superstructure for the case where the superstructure is driving into the retained soil. This is intended to cover the case where the elastomeric bearings are assumed to slip. The abutments shall be designed for the forces induced by shear deformation of the elastomeric bearings from the displacement of the superstructure under seismic loading and this load shall be applied in the direction assuming the superstructure is driving away from the retained soil. This is intended to cover the case where the bearings are assumed not to slip.

If the superstructure is positively restrained in the longitudinal direction at the piers with shear keys (or anchor bolts if allowed) so that the superstructure cannot displace more than the restraining pier, the abutment backwall need not be designed for seismic forces if the gap between the bridge superstructure and the backwall is greater or equal to 2 times the calculated longitudinal seismic displacement of the restrained superstructure. The abutments shall still be designed for seismic forces as distributed according to the first paragraph above.

3.4.8.2 Seismic Isolation, Type 3 SDS. For all bridges, isolation bearings shall be designed in accordance with the latest *AASHTO Guide Specifications for Seismic Isolation Design*. True isolation bearings shall be designed to permit the superstructure to undergo the calculated seismic displacements without restraint from the substructure and shall act as energy dissipating elements. When it is deemed appropriate, it may be permitted to design conventional steel reinforced elastomeric bearings as isolation bearings. Design of these bearings as isolation bearings shall follow the requirements of the Seismic Isolation Guide Specifications.

3.4.8.3 Partial Seismic Isolation, Combined Type 1 and Type 3 SDS. PTFE bearings designed with sliding surfaces that are allowed to slide during a seismic event shall be modeled as true frictionless bearings. However, the substructures under the sliding bearings shall still be checked for the friction force that develops at the bearing when the superstructure slides on it during the seismic event. Although this friction force can be modeled explicitly in a refined model, this is not desired since it will reduce the design seismic forces on the restraining elements by the amount of the friction force. The true frictionless case is intended to model the situation where the superstructure experiences a vertical acceleration component in addition to the horizontal, which reduces the vertical force of the superstructure on the bearing, which, in turn, reduces the friction force.

The coefficient of friction between sliding surfaces during a seismic event is not well defined in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. In lieu of testing the bearing, the forces for checking the substructure units may be calculated as 50% of the static design coefficient of friction. The use of PTFE sliding bearings will require superstructure restraint at some substructure units to prevent a loss of support failure. Typically the use of PTFE sliding bearings for seismic isolation would be targeted to substructure elements within a bridge that are incapable of resisting the seismic loads and redistributing them to other more robust substructure elements that are more capable of resisting the loads, for example, in a bridge rehabilitation project isolating slender piers and redistributing the seismic loads to the abutments.

3.4.8.4 Unique Bearings, Type 1 and/or Type 3 SDS. Typically for large-scale proprietary bearings, such as multi-rotational disc bearings, the bearing to be constructed shall be designed by the manufacturer chosen by the Contractor. Also, these types of bearings are typically used in larger bridge structures. The choice of fixed, expansion, sliding or isolation bearings shall be established as part of the overall bridge SDS and the thermal expansion/contraction requirements of the bridge structure. In these cases, the final bearing configurations will not be fully known at the time of the design. Therefore, to permit the completion of the design, bearing manufacturers shall be contacted for seismic performance and expansion characteristics to be used and that information incorporated into the seismic analysis and substructure design.

3.4.9 Seismic Design of Wingwalls and Retaining Walls

3.4.9.1 For wingwalls and free-standing retaining walls classified as SDC A or B a seismic analysis is not required. For walls classified as SDC C or D, the seismic soil forces shall be used.

3.4.9.2 Walls, including MSE and other wall types, that provide direct support to a bridge stub abutment shall be designed for the seismic soil forces acting on the wall and the inertial forces of the wall as well as the superstructure seismic forces that are transmitted from the stub abutment, regardless of SDC. For all SDCs, this superstructure seismic force shall be equal to the acceleration coefficient, A_S, times the tributary permanent load of the superstructure. For example, for a multi-span bridge where there is a longitudinal shear key at the pier to help share the longitudinal seismic force, this tributary permanent load would extend to the midpoint of the span from the abutment to the pier. This more stringent requirement is applied to these walls because the *AASHTO LRFD* specify that a wall shall be designed for seismic loads if it provides support to a structure that has to be designed for seismic loads. The reasoning behind this requirement is that if this wall suffers a failure or partial failure in a seismic event, this will compromise the bridge structure that is supported by that wall.

If the MSE or other wall type retains the embankment soil only and the bridge abutment has an independent foundation that does not rely on it for support (such as in the case of pile supported abutment or an abutment sitting on drilled shafts), then these walls do not have to be designed for seismic loads.

3.5 SUPERSTRUCTURE DESIGN

3.5.1 Composite Design

3.5.1.1 All stringer bridges will be designed compositely with the deck. All beams shall be designed for composite action without the use of temporary intermediate supports during the placing and curing of the deck concrete. Composite section properties shall be calculated based on the short-term modular ratio (n) or long-term modular ratio (3n), where:

$$n = \frac{E_B}{E_C}$$

In the above formula, E_B is the Modulus of Elasticity of the beam material (either steel or prestressed concrete), and E_C is the Modulus of Elasticity of the cast-in-place concrete deck.

3.5.1.2 When calculating any composite section properties, the depth of the standard haunch as detailed in Part II of this Bridge Manual shall conservatively be assumed to be zero. This is due to the fact that actual depth of the haunch varies depending on the amount of over-cambering in the beam.

3.5.1.3 For steel beams, when calculating stresses due to dead loads acting on the composite section, the effect of creep will be considered by using the long-term modular ratio. For prestressed concrete beams, the same composite properties shall be used for calculating both superimposed dead load and live load stresses.

3.5.1.4 For the design of continuous beams, MassDOT requires that, at a minimum, the total cross-sectional area of the longitudinal reinforcement provided in the negative moment regions, between points of dead load contraflexure, shall not be less than 1% of the total cross-sectional area of the concrete deck, regardless of the calculated deck concrete stress levels. This minimum area of the longitudinal reinforcement shall include the area of both top and bottom longitudinal distribution reinforcement layers already placed in the deck and any additional reinforcement needed to get to required minimum 1%. The maximum size of the additional longitudinal reinforcement should not exceed No. 6 bars. The spacing of individual bars should not exceed 12”. The negative flexure deck longitudinal reinforcement shall be placed and distributed according to the *AASHTO LRFD*.

Please note that an area of longitudinal reinforcement ***in excess of the minimum 1%*** may need to be provided if the Designer determines that the deck longitudinal tensile stress due to factored construction loads or Load Combination Service II exceeds the limit specified in *AASHTO LRFD*.

The Designer should also be mindful of the commentary to Article 6.10.1.7 of the *AASHTO LRFD*, which advises that the above provisions for cross-sectional area of longitudinal reinforcement may also need to be applied to the deck areas outside of the negative moment regions, wherever the tensile concrete deck stress due to the factored construction loads, loads during the various phases of the deck placement, or due to Load Combination Service II exceed the limit specified in the Article.

3.5.1.5 Stud shear connectors shall be used for composite steel beams. The pitch of the studs need not be made in multiples of the spacing of transverse steel reinforcement in the deck slab and it should be based on fatigue requirements. The total number of studs provided must be adequate for the strength limit state requirements in accordance with the *AASHTO LRFD*. For continuous beam design it is the policy of MassDOT that stud shear connectors shall be used throughout the length of the continuous composite beams in the positive and negative moment regions.

3.5.1.6 When designing continuous composite steel girders for Strength Limit States, the moments along the girder length shall be distributed assuming gross section properties of the composite girder; however, in the negative moment region, the composite section consisting of the steel girder and the longitudinal reinforcement within the effective width of the concrete deck only, shall be used to check the girder.

3.5.1.7 Appendices A6 and B6 of the *AASHTO LRFD*. Due to the narrow range of application, the provisions of Appendix A6 to determine the flexural resistance of straight composite sections in negative flexure should not be used for MassDOT projects. However, Designers may use the design procedure of Appendix B6 for the redistribution of moment from the negative to the positive moment regions.

3.5.1.8 If the superstructure is comprised of simple span prestressed beams made continuous for live load, it should be designed according to Article 5.12.3.3 of the *AASHTO LRFD*. For any of the prestressed beams detailed in Part II of this Bridge Manual, the design for end restraint moment can be neglected, however the bottom strands shall be extended as shown in Part II. For simple span steel beams made continuous for live load see Subsection 3.6.6.

3.5.1.9 Prestressed concrete beams designed compositely shall use dowels casted into the beams and subsequently embedded into the deck slab to transfer the horizontal shear. These dowels shall be detailed as shown in Part II of this Bridge Manual and shall be designed in accordance with the *AASHTO LRFD* requirements for shear-friction (interface shear) for composite flexural members.

3.5.2 Deck Slabs

3.5.2.1 Steel reinforcement and deck slab thickness shall be as per the design tables of Chapter 7 of Part II of this Bridge Manual. If the beam spacing falls outside of the table limits, the deck slab reinforcement shall be designed using the traditional approximate method of analysis identified in *AASHTO LRFD* Article 9.7.3, not the empirical deck design method shown in Article 9.7.2. The deck shall be treated as a continuous beam. Moments as provided in *AASHTO LRFD* Table A4-1 are to be applied at the design sections identified in Article 4.6.2.1.6 of the *AASHTO LRFD*. A Designer may also perform an analysis in accordance with Article 4.6.2.1.6 of the *AASHTO LRFD,* in lieu of using the Chapter 7 tables in Part II of this Bridge Manual.

Steel reinforcement for the deck slab overhangs shall be as per the design tables of Chapter 7 of Part II of this Bridge Manual. If the deck slab overhang exceeds the limits specified in the tables, the Designer shall design the deck reinforcement in accordance with Section 13 of the *AASHTO LRFD* for the given test level of the railing/barrier system.

All deck reinforcement shall be coated (either epoxy coated or galvanized).

3.5.2.2 All CIP Deck slabs with or without hot mix asphalt wearing surface shall be constructed using high performance concrete (HPC). Decks without membrane waterproofing and hot mix asphalt wearing surface shall be constructed in one single full-depth placement. The top ¾” of such placements shall be considered sacrificial and shall not be used when calculating the section properties.

3.5.2.3 Membrane Waterproofing for Bridge Decks (Item 965) shall be the only waterproofing membrane used for all new CIP Deck Slabs as well as for deck systems that are constructed from PBU, NEXT D beams, NEDBT beams, and Precast Concrete Full Depth Deck Panels. A system consisting of the Membrane Waterproofing for Bridge Decks with an aggregate key coat, a polymer modified tack coat, and a hot mix asphalt wearing surface shall be used on all bridges with a profile grade up to and including 6%, except for those bridges with profile grades between 4% and 6% that immediately abut an intersection where heavy trucks must routinely either slow down to a stop or start to turn on the bridge, thereby generating large shear forces in the asphalt. For these locations, the Designer shall consult with the State Bridge Engineer and the State Pavement Engineer to determine the most suitable deck wearing surface.

3.5.2.4 Stay-in-place (SIP) forms shall be used for deck construction over rivers, active railroad tracks and roadways that will remain open to the public during construction, except as noted below.

The locations on typical bridge decks where SIP forms are prohibited and removable forms shall be used are as follows:

- In the deck bays, full length of the bridge, which are directly under the curb or barrier lines.
- In the deck bays, full length of the bridge, where there are longitudinal stage construction joints.
- For the forming of end diaphragms and overhanging portions of the deck slab.
- At the locations of scuppers and downspouts.
- Within a distance of 4' on both sides of the deck transverse construction joints.

3.5.2.5 Top-of-form elevations must be provided in order to set the forms such that, after all dead loads have been applied, the top of roadway will be at the correct profile elevation. Top-of-form elevations will be calculated as follows:

1. Calculate the theoretical top of roadway elevation directly over the beam at the required points along its span as specified in Part II of this Bridge Manual.
2. From this elevation, subtract the thickness of the wearing surface and deck to obtain the in-place bottom of deck elevation, neglecting the thickness of the membrane waterproofing.
3. To the in-place bottom of deck elevation, add the total dead load deflection of the beam, excluding the deflection due to the beam's self-weight, calculated for the particular point along the beam under consideration. The result is the top-of-form elevation.

This is required for all structures comprised steel or prestressed girders using forms between the girders to support the concrete for the deck. Use the PCI at erection multipliers to calculate the deflections to be used in these calculations.

3.5.2.6 For decks that are cast on adjacent prestressed concrete beam bridges or NEXT F beam bridges, where the top of the beam is the deck form, top of deck elevations shall be provided instead of top-of-form elevations. The top of deck elevations shall be calculated similar to the top-of-form procedure outlined in Paragraph 3.5.2.5 except that in step 2, the thickness of the deck shall not be subtracted from the top of roadway elevation, only the wearing surface. Top of deck elevations shall be provided at beam tenth points along the curb lines and at the crown. These top of deck elevations will also be used to ensure that any reinforcing extending from the beams or the deck, such as barrier reinforcing, is detailed to an adequate length.

3.5.2.7 For NEDBT and NEXT D beam bridges, where top-of-form or top of deck elevations are not required since the top of the beam is also the top of the deck, the Designer shall provide the theoretical top of curb form elevations at the bearings and the span tenth points. The theoretical top of curb elevations shall be calculated by taking the proposed top of HMA elevation at the curb line and adding to it the curb reveal dimension and the superimposed dead load deflection of the beam.

3.5.2.8 Link slabs may be used to eliminate deck joints at piers where each span is supported on elastomeric bearings without anchor bolts. A link slab is comprised of a reinforced concrete deck with a length that extends approximately 5% to 7% of each adjacent span length from the centerline of the pier, not necessarily the same percentage for each span. Shear stud connectors shall be omitted within the limits of the link slab and a bond breaker is applied between the top flange and the link slab to prevent composite action and improve the flexibility of the link slab. The total number of shear stud connectors per span required to meet strength requirements shall still be provided beyond the link slab region. The shear studs in the remaining portion of the span shall meet the number required for ultimate strength. Where applicable, the Designer should consider the placement of paraffin joints in sidewalks and barriers over the gap between the beam ends.

Link slabs may also be used for bridge rehabilitation projects provided the Designer investigates the impact of the overall thermal movement with the new span configuration. This may require modifications or replacement of the existing bearings.

The link slab reinforcing is designed as outlined in the design procedure that follows to minimize crack widths based on the anticipated strains due to live load rotations for an interior girder. If required by design, transition zones adjacent to the link slab shall provide a tension lap splice between the link slab longitudinal reinforcement and the deck longitudinal reinforcement.

General design and detailing provisions and assumptions:

- Bridge skew can be neglected in the design of link slabs.
- For cast-in-place construction, the concrete used for the link slab will be the same concrete used for the connected decks. For PBUs or prestressed beams with integral decks that are connected with link slabs, the closure pour concrete may be used for the link slab.
- The thickness of the link slab shall be equal to the connected decks. No thickening of the deck within the link slab region is allowed.
- Design the transverse reinforcing steel in the link slab, including deck overhangs using the same methods as the connected decks.

Link Slab Design Procedure:

1. Design adjacent spans as simply supported neglecting the proposed link slab.
2. The length of the link slab L_{ls} shall be the summation of 5% to 7% of each span length plus the distance between the beam ends at the pier. The link slab shall be debonded from the girders and no shear stud connectors shall be provided in this region. (See Figure below).

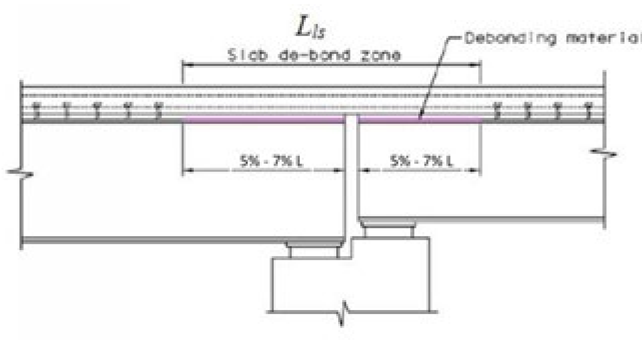


Figure 3.5.2-1: General Link Slab Layout

3. Determine the end rotations of the girders Q, from the beam design under service (unfactored) live loads. If the software used does not provide end rotations, the end rotations can be determined using the midspan deflections. The end rotation may be estimated as equal to $3.2*\Delta_{LL} / L$ where L = Span length of the girder and Δ_{LL} = mid span deflection of the girder. The deflection used corresponds to the controlling end rotation, Q.
4. Calculate the moment of inertia of the link slab $I_{ls} = b*t^3/12$ where b = composite width of the deck and t = thickness of the link slab
5. Calculate the negative moment in the link slab, $M_{ls} = 2E_{ls}I_{ls}Q / L_{ls}$ where E_{ls} is the modulus of elasticity of the link slab concrete.
6. Calculate cracking moment of link slab, M_{cr} (*AASHTO LRFD,* Eq. 5.6.3.5.2-2).
7. If $M_{ls} < M_{cr}$ then normal deck distribution reinforcement can be used as the longitudinal reinforcement in the link slab.
8. If $M_{ls} > M_{cr}$ then cracks can be expected in the link slab and additional reinforcement is required.
9. Design the reinforcement for the link slab to resist the applied moment using working stress methods and check the control of cracking criteria per *AASHTO LRFD.* Use γ_e, of 0.75 for Class 2 exposure condition. The tensile stress in the reinforcement, f_{ss}, shall not exceed $0.4F_y$.

3.5.3 Distribution of Loads on Stringer Bridges

3.5.3.1 General. The purpose of this Subsection is to establish consistent MassDOT procedures for distributing loads to the beams of stringer type bridges. The provisions of this Subsection apply to all types of stringer bridges, except as modified in Section 3.8 below for adjacent beam bridges.

First, this Subsection outlines procedure for distributing sidewalk and barrier dead loads to beams using a pile cap analogy. Historically, the AASHTO bridge design specifications have specified equal distribution of sidewalk and barrier dead loads to all beams in the cross section. Since the 1970's, Massachusetts has applied 60% of these loads to their immediate supporting beams. This distribution is based on the realization that these beams will see more of the superimposed dead load of a sidewalk or barrier that is cast on the edge of a slab than would the interior beams, especially on wide bridges. The value of 60% was not based upon any detailed analysis or study but rather upon engineering judgment.

However, in preparing this Subsection, MassDOT realized that the 60% load distribution formula was too simplistic and could not be easily adapted for cases where there was more than one beam under the sidewalk or if the first interior roadway beam was close to the curb. As a result, MassDOT undertook a grillage analysis study of typical bridges with various beam spacings and span lengths in order to calculate the actual sidewalk slab dead load distribution to the beams. Based on the results of this study, MassDOT found that the pile cap analogy provided a reasonable estimate of the actual sidewalk slab load distribution. Because this method is already presented in the *AASHTO LRFD* for the distribution of live load to exterior beams and is relatively simple to use, MassDOT decided to adopt this load distribution methodology to replace the 60% method.

For sidewalks, this distribution will extend to the first (or more, in the case of a wide sidewalk) interior roadway beam which will be used as the basis for the design of all interior beams in order to bracket the potential load effects for narrow bridges, where the interior beams could see more of these superimposed dead loads. The pile cap analogy will also address the effect of overhang length on load distribution.

Second, this Subsection also provides a methodology for distributing pedestrian live loads to the beams, since the *AASHTO LRFD* do not have specific design procedures. The MassDOT procedures incorporate the procedures from the AASHTO commentary along with specific clarification in their application.

The provisions of this Subsection assume a typical MassDOT highway bridge and they neglect the beneficial effect of stiffening due to the sidewalk slabs and/or barriers. It is conservative when compared to the AASHTO provisions, which require that all loads placed after the deck slab is cured be equally distributed to all beams.

3.5.3.2 Design Procedure. The intent of these provisions is that the exterior beams should meet all loading situations that they may be reasonably expected to see without necessitating beam sections that are markedly larger than those of a typical interior beam. However, in no case shall the exterior beam have less non-composite section than an interior beam.

Therefore, the following steps outline the general procedure:

1. Design the first interior roadway beam.
2. Use the same beam section to check the other interior beams.

3. Use the same beam section to check the exterior sidewalk/safety curb beam, revise section if necessary.

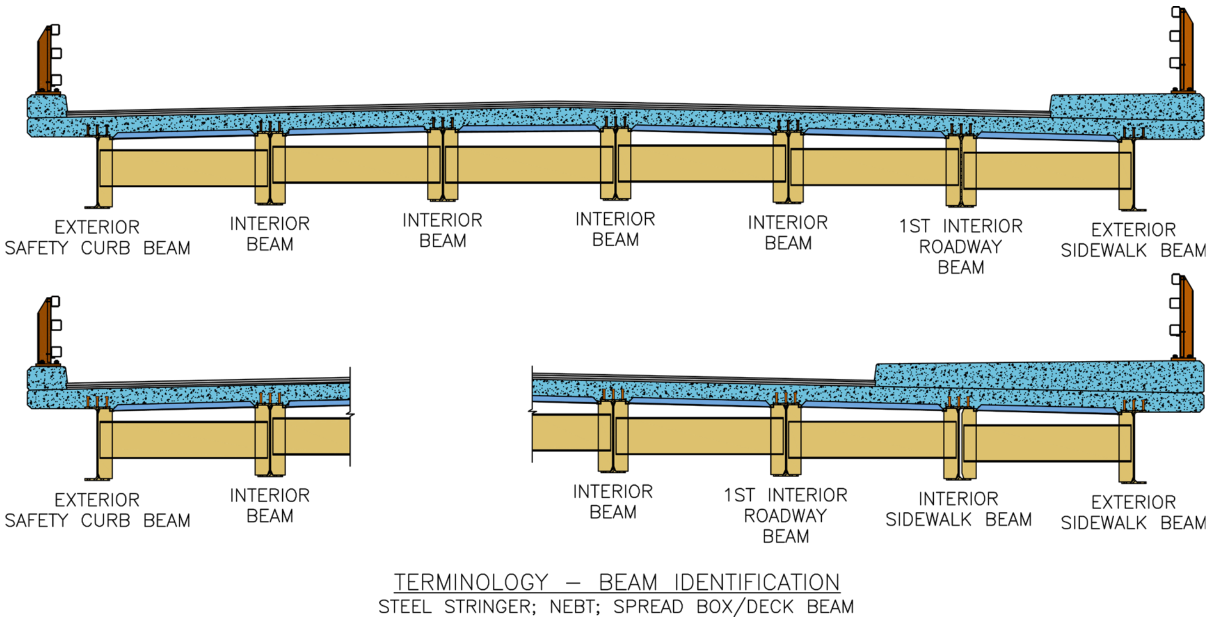


Figure 3.5.3-1: Terminology for Bridge Beam Identification

3.5.3.3 Non-Composite Dead Load Distribution (DC1). The non-composite dead loads, in addition to the beam self-weight, shall include the diaphragms or cross frames, utilities and other attachments, the deck and the deck haunch, and the additional concrete of the soffit at the exterior beams, which extends out over the entire overhang. For all beams, the deck slab dead load shall be distributed to each beam directly below based on tributary area. Utility loads can generally be assumed to be non-composite dead loads that are equally distributed to the beams that support them on either side of the utility bay.

3.5.3.4 Superimposed Dead Load Distribution (DC2 & DW)

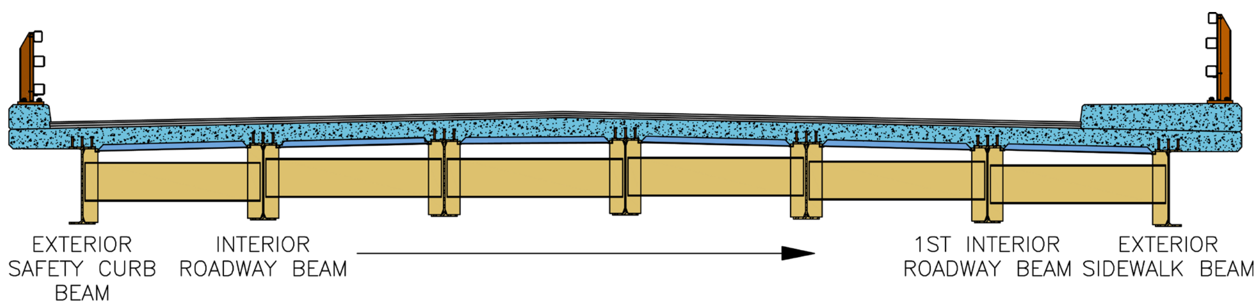
1. For the first interior roadway beam, the wearing surface superimposed dead load shall be distributed to it by dividing this load by the total number of the beams (interior and exterior) in the cross section. The sidewalk slab and barrier/railing superimposed dead loads shall be distributed to the beam using the pile cap analogy (refer to Figure 3.5.3-2 and Figure 3.5.3-3 below).

2. For interior beams (other than the interior sidewalk beam), the wearing surface, sidewalks, safety curbs, and barriers/railings superimposed dead loads shall be distributed equally to all beams, i.e. sum of these loads divided by the total number of beams (interior and exterior) in the cross section (refer to Figure 3.5.3-2 and Figure 3.5.3-4 below).

3. For the exterior beam supporting a safety curb or barrier, the wearing surface superimposed dead load shall be distributed dividing this load by the total number of the beams in the cross section (interior and exterior) and the safety curb/barrier/railing shall be distributed to the beam supporting the safety curb using the pile cap analogy (refer to Figure 3.5.3-2 and Figure 3.5.3-5 below).

4. For the exterior sidewalk beam, the wearing surface superimposed dead load shall be distributed to it by dividing this load by the total number of the beams (interior and exterior) in the cross section. The sidewalk slab and barrier/railing superimposed dead loads shall be distributed to the beam using the pile cap analogy. If the sidewalk is supported by more than one beam, the superimposed dead loads (wearing surface, sidewalk slab and railing/barrier) shall be distributed to each of these beams as outlined above for the exterior sidewalk beam (refer to Figure 3.5.3-2 and Figure 3.5.3-6 below).

3.5.3.5 Pedestrian Load (PL) Distribution. For interior beams and exterior safety curb beams, the distribution of the Pedestrian Live Load shall be similar to that of the superimposed dead loads, i.e. total pedestrian live load divided by the total number of beams in the cross section (interior and exterior). For the exterior sidewalk beam (and interior sidewalk beams) and the first interior roadway beam, the Pedestrian Live Load shall be distributed using the pile cap analogy, (refer to Figure 3.5.3-2 below). The Dynamic Allowance Factor (IM) shall not be applied to Pedestrian Live Loads. When designing continuous beams, the Pedestrian Live Load shall be positioned along the span in such manner, as to produce the maximum load effect in the beam.



LOAD DISTRIBUTION

SAFETY CURB	INTERIOR		1ST INTERIOR	SIDEWALK
TRIBUTARY	TRIBUTARY	DECK:	TRIBUTARY	TRIBUTARY
EQUAL	EQUAL	WEARING SURFACE:	EQUAL	EQUAL
NA	EQUAL	SIDEWALK:	PILE CAP	PILE CAP
NA	EQUAL	SIDEWALK BARRIER:	PILE CAP	PILE CAP
PILE CAP	EQUAL	SAFETY CURB:	NA	NA
EQUAL	EQUAL	PEDESTRIAN LL:	PILE CAP	PILE CAP

Figure 3.5.3-2: Load Distribution to Bridge Beams

3.5.3.6 Pile Cap Analogy. The application of pile cap analogy is simplified method to determine load distribution using the well-known formula of $\frac{P}{A}+\frac{M}{S}$; Typically, in a pile group calculation, the term of $\frac{M}{S}$ can be both positive and negative (+ or -). In this case however, for the purpose of distributing superimposed dead loads, the uplift from the loads applied shall not be used to reduce the load effects on the beams on the other side of the center of gravity of the group.

In the calculation of the center of gravity of the group and its "inertia", the actual individual stiffness of the beams should typically be ignored, i.e. A = 1.0 for all beams.

3.5.3.7 Multiple Presence Factor (m) and Pedestrian Load. According to the *AASHTO LRFD*, Article 3.6.1.1.2, when Pedestrian Live Load is combined with one or more lanes of vehicular live load, it may be considered as one loaded lane for the purpose of determining the Multiple Presence Factor. However, this use of the Pedestrian Live Load as a loaded lane shall only apply to the design of exterior beams or other interior sidewalk beams and not for the design of interior roadway beams, even though for design purposes, part of the Pedestrian Load is applied to them.

Furthermore, the *AASHTO LRFD,* Commentary C3.6.1.1.2 states that the Multiple Presence Factor of 1.20 for a single lane does not apply to the pedestrian loads. Therefore, the Multiple Presence Factors for different combinations of vehicular live load and pedestrian load shall be as follows:

- Pedestrian Live Load only, m = 1.00
- One traffic lane only, m = 1.20
- Pedestrian Live Load and one traffic lane, m = 1.00
- Two traffic lanes only, m = 1.00
- Pedestrian Live Load and two traffic lanes, m = 0.85
- Three traffic lanes only, m = 0.85
- Pedestrian Live Load and three traffic lanes, m = 0.65
- More than three traffic lanes with or without Pedestrian Live Load, m = 0.65

The *AASHTO LRFD,* Article 3.6.1.1.2 states that these Multiple Presence Factors shall not be applied in conjunction with the approximate load distribution factors specified in Article 4.6.2.2.1, but they are to be applied when using the lever rule or the pile cap analogy to distribute the HL-93 Live Load.

3.5.3.8 **First Interior Roadway Beam** - Distribution of Loads for the Design (see Figure 3.5.3-3 below):

1. All Non-Composite Dead Loads, DC1, as per Paragraph 3.5.3.3.
2. Superimposed Dead Load, DC2 & DW, as per Paragraph 3.5.3.4 (1).
3. Pedestrian Load, PL, as per Paragraph 3.5.3.5.
4. The entire HL-93 live load plus dynamic load allowance (IM) shall be distributed to first interior beam using the Distribution Factors from the procedures outlined in the *AASHTO LRFD*, Article 4.6.2.2.2.

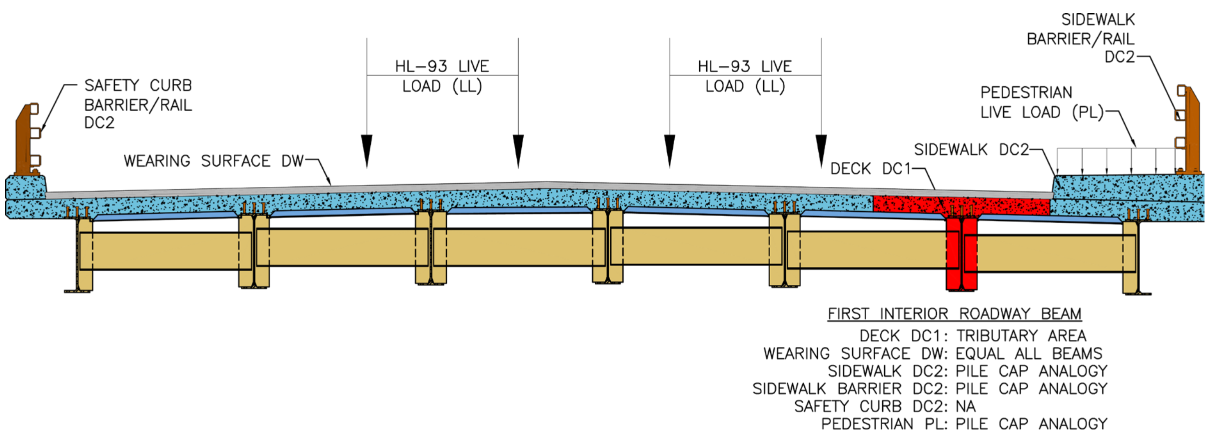


Figure 3.5.3-3: Distribution of Loads to First Interior Beam

3.5.3.9 **Interior Beams** - Distribution of Loads for the Design (refer to Figure 3.5.3-4 below):

1. All Non-Composite Dead Loads, DC1, as per Paragraph 3.5.3.3.
2. Superimposed Dead Load, DC2 & DW, as per Paragraph 3.5.3.4 (2).
3. Pedestrian Load, PL, as per Paragraph 3.5.3.5.

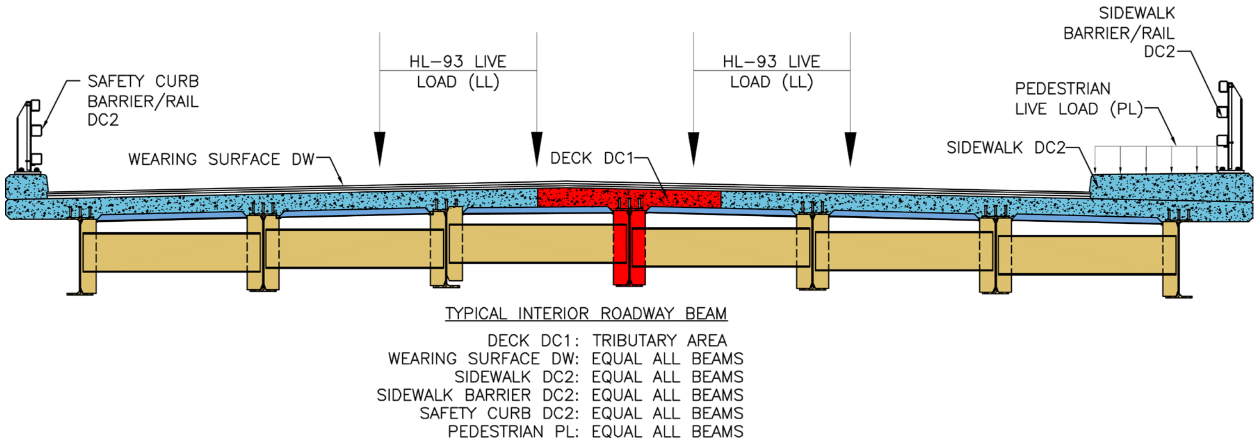


Figure 3.5.3-4: Distribution of Loads to Interior Beams

The entire HL-93 live load plus dynamic load allowance (IM) shall be distributed to interior beams using the Distribution Factors from the procedures outlined in the *AASHTO LRFD*, Article 4.6.2.2.2.

3.5.3.10 **Exterior Beams under Safety Curbs or Barriers** - Distribution of Loads for the Design (refer to Figure 3.5.3-5 below):

1. All Non-Composite Dead Loads, DC1, as per Paragraph 3.5.3.3.
2. Superimposed Dead Load, DC2 & DW, as per Paragraph 3.5.3.4 (3).
3. Pedestrian Load, PL, as per Paragraph 3.5.3.5.
4. The entire HL-93 live load plus dynamic load allowance (IM) shall be distributed to the exterior beam using the Distribution Factors from the procedures outlined in the AASHTO LRFD, Article 4.6.2.2.2.

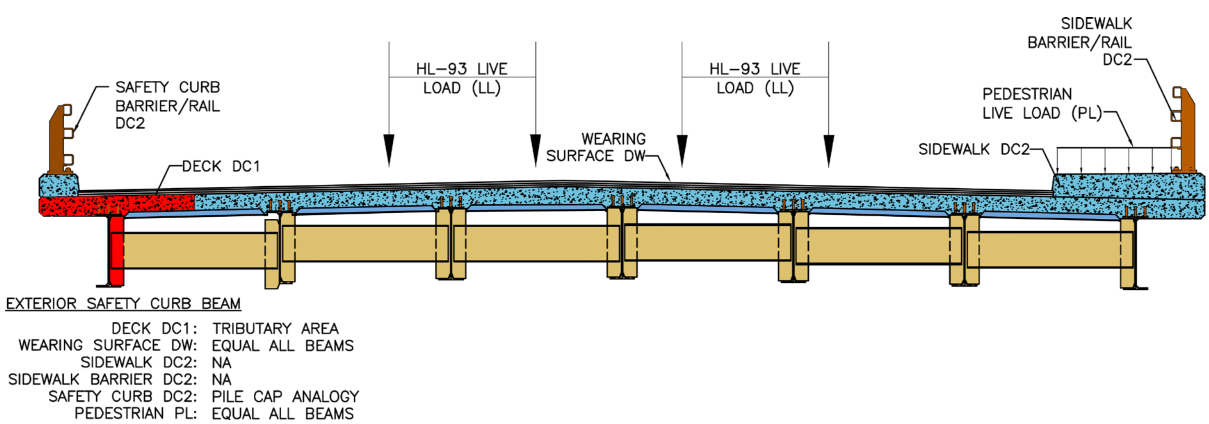


Figure 3.5.3-5: Distribution of Loads to Exterior Beams under Curbs

3.5.3.11 **Exterior Beams under Sidewalks** - Distribution of Loads for the Design.

Case I. The exterior beam under the sidewalk shall be checked/designed according to the following design load case (refer to Figure 3.5.3-6 below).

1. All Non-Composite Dead Loads, DC1, as per Paragraph 3.5.3.3.
2. Superimposed Dead Load, DC2 & DW, as per Paragraph 3.5.3.4 (4).
3. Pedestrian Load, PL, as per Paragraph 3.5.3.5.
4. The HL-93 Live Load plus Dynamic Load Allowance (IM) shall be distributed to the exterior beam under the sidewalk according to the procedures outlined in the *AASHTO LRFD*, Article 4.6.2.2.2d. The HL-93 Live Load(s) shall be located starting 2 feet from the face of the sidewalk curb and shall be placed in the design lanes across the bridge as specified in the *AASHTO LRFD*. Since for typical MassDOT sidewalk geometries the exterior beam is located further away from the roadway traffic lanes, it will not see as much of the load as calculated using the approximate load distribution factors specified in Article 4.6.2.2.2d. To avoid these overly conservative live load effects, only the lever rule or pile cap analogy shall be used and the Multiple Presence Factor applied.

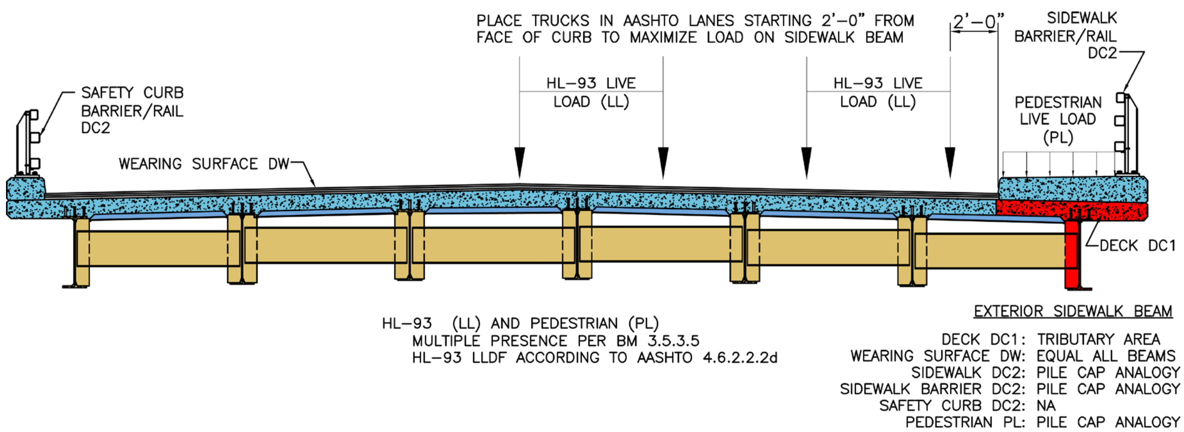


Figure 3.5.3-6: Distribution of Loads to Exterior Beams under Sidewalks – Case I

Case II. The exterior beam under the sidewalk shall be checked for a truck on the sidewalk as follows (refer to Figure 3.5.3-7 below).

1. Distribute all Dead Loads as per Design Case I above.
2. Do not apply the Pedestrian Load.
3. Using the Strength II Limit State, apply the truck load portion of a single HL-93 Live Load (no other lanes are loaded) ***without*** Dynamic Load Allowance (IM) located 2 feet from the face of the barrier/railing, as if the sidewalk was not there and distribute it to the exterior beam, or beams if more than one beam supports the sidewalk, using the pile cap analogy and a multiple presence factor m = 1.20.

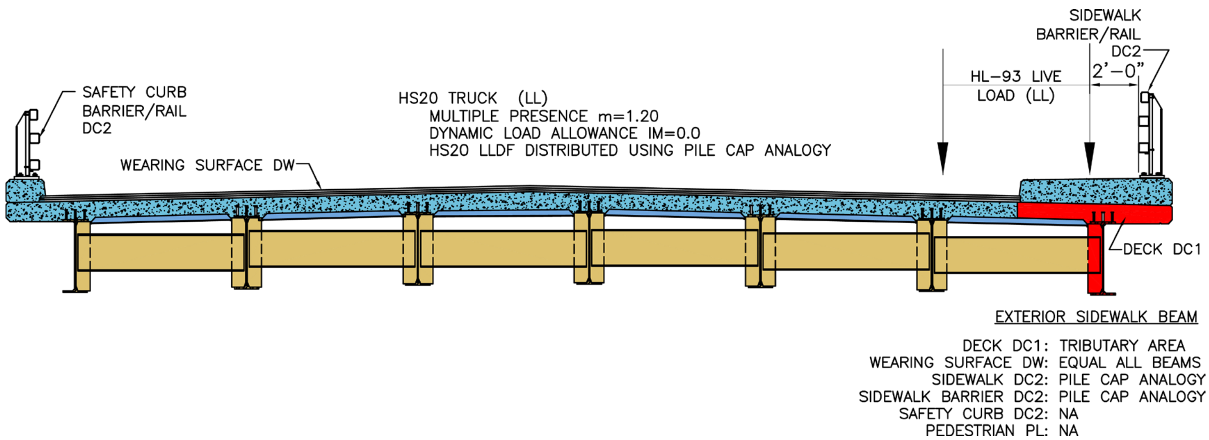


Figure 3.5.3-7: Distribution of Loads to Exterior Beams under Sidewalks – Case II

Note that a strict adherence to the *AASHTO LRFD*, Article 4.6.2.2.2d would require checking the Live Load distribution using Lever Rule provision. However, the Lever Rule is inherently conservative, because, in reality, the imaginary hinge will not form in the deck and it will remain capable of distributing the Live Load across to the other beams. Furthermore, the additional stiffness provided by the sidewalk slab is not considered in the analysis, and the remote likelihood of a truck being on the sidewalk does not warrant an overly conservative design approach. Therefore, for simplicity, only the pile cap analogy is specified here for this load case.

3.5.3.12 All beams under a raised median with a continuous roadway slab shall be designed as interior beams in accordance with Paragraph 3.5.3.9. If there is a longitudinal joint in the median so that the roadway slab is discontinuous, then the beams adjacent to this joint shall be designed as exterior beams in accordance with Paragraph 3.5.3.10.

3.5.4 Distribution of Loads on NEXT Beam Bridges

3.5.4.1 The distribution of loads and load cases as specified in Subsection 3.5.3 shall also apply to NEXT beams.

3.5.4.2 Non-Composite Dead Loads (DC1), Superimposed Dead Loads (DC2 & DW), Pedestrian Load (PL) shall be distributed to NEXT beams treating each stem of the beam as an individual stringer. The computed loads for each of the stems shall be combined and the resulting load shall be applied to the entire NEXT beam.

3.5.4.3 Vehicular Live Load Distribution Factor (LLDF) shall be established for each NEXT Beam as outlined below. When applying the *AASHTO LRFD* Table 4.6.2.2.1-1, use cross section Type (k) for NEXT F Beam Bridges and Type (i) for NEXT D Beam bridges, since the closure pours between NEXT D beams are sufficient to connect individual beams to act as a unit.

1. Treat each stem as an individual stringer, and calculate the LLDF for each stem using Table 4.6.2.2.2b-1 and Table 4.6.2.2.2d-1 of the *AASHTO LRFD* for interior and exterior stems, respectively. For the spacing parameter, S, in the AASHTO formulas, use an equivalent stem spacing, S_{eq}, which is equal to one half of the distance between the NEXT Beam centerlines (i.e. $S_{eq} = S_{BMCL}/2$). See Figure 3.5.4-1. Refer to Paragraph 3.7.6.1 of this Bridge Manual for the determination of the Longitudinal Stiffness Parameter, Kg.

2. For exterior stems, the application of the lever rule and pile cap analogy should be investigated using the actual stem spacing and distance from the exterior edge of the flange/deck to the centerline of the exterior stem.
3. Add the LLDFs calculated above for each of the two stems that comprise the NEXT Beam to arrive at the LLDF to be used for the design of the NEXT Beam itself.

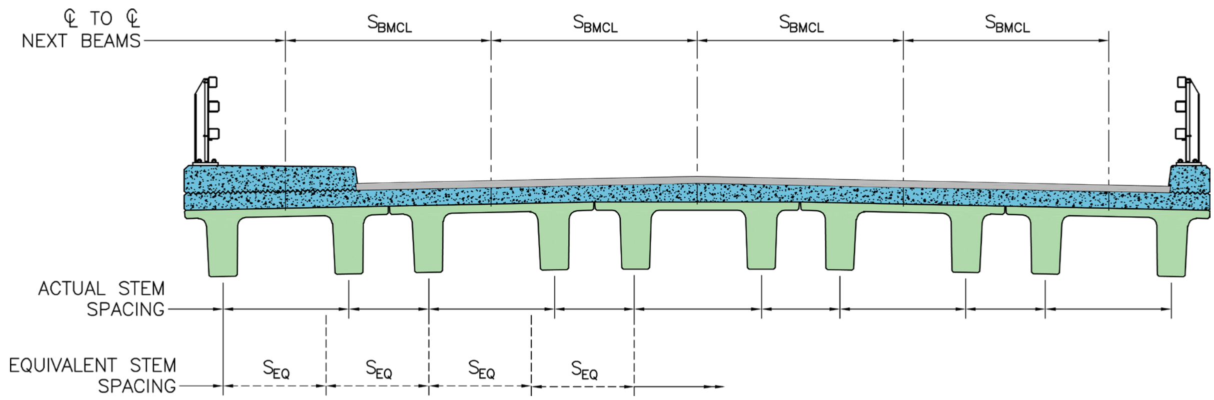


Figure 3.5.4-1: Equivalent NEXT Beam Stem Spacing Example

3.5.4.4 When designing NEXT D Beams, the following shall be noted: if a precast or cast-in-place barrier is used, it should be placed after the longitudinal joint pours are made to allow for the distribution of the parapet superimposed dead load as per procedure specified above. If the barrier is placed before the longitudinal joint pours are made, the dead load of the parapet shall be applied entirely to the exterior beam below it.

3.5.5 Utilities on Structures

3.5.5.1 Typical details for utility supports for the various types of superstructures are shown in the Part II of this Bridge Manual. At the initiation of the project, the Designer shall investigate and identify all utilities (existing and/or proposed) carried on the structure or crossing its footprint. The Designer shall submit to the MassDOT Utility/Railroad Engineer letter(s) of transmittal that the said utility investigation was performed, and resolution of all issues was achieved. All existing and proposed utilities shall be shown on the Construction Drawings. Railroads may have additional utility placement requirements that the Designer shall incorporate in the design.

3.5.5.2 All utilities on stringer bridges shall be carried in the utility bay(s) of the superstructure and shall be accessible from below. When the number and type of utilities that will be carried by the bridge is known at the time of design, the actual utility loads may be used for all design calculations including beam camber and the beam dead load deflection for top-of-form elevations. The dead load of utilities is assumed to be carried by the two beams on either side of the utility bay. Utilities shall not be embedded within a deck slab or sidewalk slab because their presence there could inhibit future maintenance activities.

When replacing bridges carrying local roads with no existing utilities present, a utility bay shall be provided on the new structure. In these situations, the stringers on either side of the utility bay shall be designed for a future utility load of 125 pounds per foot per beam (for a combined utility load of 250 pounds per foot per bay). Since this load already accounts for some uncertainty of the utility type and its load, the maximum load factor γ_p shall be taken as 1.25. Also, since these utilities may never be installed, the minimum load factor γ_p shall be taken as 0.0. Furthermore, these future utility loads shall not be used in calculating beam camber or the beam dead load deflection for top-of-form elevations.

For bridges carrying interstate or other limited access highways, provisions for fiber optic and highway lighting conduits shall be made.

3.5.5.3 Utilities on adjacent deck and box beam bridges shall be carried and designed for as specified in Subsection 3.8.2.

3.5.5.4 Utilities are typically installed before the deck is placed since it facilitates their installation and alignment both horizontally and vertically. Therefore, the non-composite section shall carry the total dead load of utilities.

3.5.5.5 When the utility is to be installed for a municipality, such as a water pipe, the complete support system shall be included as part of the contract. Other utilities not installed by the Contractor, such as telephone ducts and gas mains, shall be indicated on the Construction Drawings as to their location in the utility bay or other designated area with the notation: TO BE INSTALLED BY OTHERS. The Designer is cautioned to provide utility bays of sufficient size to accommodate the utility installation.

3.5.6 Deflection and Camber

3.5.6.1 The ratio of live load plus dynamic load allowance deflection to span length shall not be greater than 1/1000 for all bridges with sidewalks. For bridges without sidewalks, this ratio preferably should not exceed 1/1000. However, under no circumstances shall it be greater than 1/800. This deflection check shall only be applied to the typical interior beam. The deflection calculation shall be performed in accordance with Article 2.5.2.6.2 of the *AASHTO LRFD*.

3.5.6.2 For steel beams, camber shall be calculated and specified on the Construction Drawings as shown in Part II of this Bridge Manual. Provide the minimum number of different camber diagrams for all beams in a given span. Group beams within a span whose maximum total camber does not vary by more than 1/8".

The amount of Additional Camber "Z", to add is given as a fraction of an inch per 10 feet of total span length between consecutive bearings. For a 100 foot simple tangent span, Z, at 1/8" per 10' of span, would calculate out to be:

Z = (100/10) x 0.125 = 1.25"

For a 200 foot total length, 2 span continuous tangent bridge consisting of two 100 foot spans, Z, at 1/16" per 10' of span, would calculate out to be:

Z = (100/10) x 0.0625 = 0.625"

3.5.6.3 For prestressed concrete beams, the net upward camber of these beams shall be calculated using the PCI "at-erection" multipliers applied to the deflections from prestressing and self-weight. The prestressing force produces moments in prestressed concrete beams that result in upward deflections. These deflections are partially offset by the downward deflections due to the beam self-weight, resulting in a net upward deflection of the beam at erection. Observation of actual bridges indicates that once the slab is placed, the prestressed concrete beams tend to behave as if they were locked in position. Prestressed concrete beam cambers shall be provided on the Construction Drawings as shown in Part II of this Bridge Manual, however they will not be used when calculating under-bridge clearances. Prestressed concrete beam cambers shall be considered along with the roadway profile vertical curvature when calculating bridge seat elevations so that the top of roadway will match the design roadway profile while the deck thickness shall not be reduced due to a negative haunch. The bridge seat elevations shall be determined using the methodology outlined in Part II of this Bridge Manual.

3.5.7 Elastomeric Bridge Bearing Assemblies

3.5.7.1 General. Elastomeric bearing assemblies shall be used for both prestressed concrete and steel beam bridges and shall be designed and fabricated in accordance with the requirements of the MassDOT Standard Specifications for Highways and Bridges, and as modified by this section.

Steel reinforced elastomeric bearing assemblies shall consist of alternate layers of steel laminates and elastomer bonded together and, either a beveled or flat sole plate for steel beam bridges, or internal load plate for prestressed concrete beam bridges, if required. Holes in either the elastomer or the steel laminates are not allowed.

3.5.7.2 Design Methodology. Method B, as presented in the *AASHTO LRFD* Article 14.7.5, is the preferred MassDOT method for designing steel-reinforced elastomeric bearings and should be used for design. Method A, as presented in *AASHTO LRFD* Article 14.7.6, may also be used to design steel-reinforced elastomeric bearings and plain elastomeric pads with the prior approval of the State Bridge Engineer, if the Designer can provide sufficient justification to support its use in place of Method B.

3.5.7.3 Design Methodology Background. In recent years, the expanded use of elastomeric bearings for steel bridges and the use of deeper beams to span longer distances have imposed greater rotational demands on steel reinforced elastomeric bearings. In previous editions of *AASHTO LRFD* the overly conservative rotational design did not allow for uplift, which unreasonably limited elastomeric bearing application or prevented their usage altogether.

Starting with the 2009 Interim Revisions to the *AASHTO LRFD* the design procedures for steel reinforced elastomeric bearings for both Method A and Method B were substantially revised. These revisions incorporate the research results and recommendations from NCHRP Project 12-68 “Improved Rotational Limits of Elastomeric Bearings”, which were subsequently included into NCHRP Report 596 “Rotational Limits for Elastomeric Bearings”.

The major changes to the design procedures incorporated in the 2009 Interim Revisions of the *AASHTO LRFD* are as follows:

- The allowable design capacities for Method A have been increased based upon the findings of the above referenced research. The fundamental form of the Method A design equations remained unchanged, increased neoprene compression capacities are provided and the check on rotational capacity (“no lift-off”) is eliminated.
- The revised procedure for Method B is based upon the actual failure mechanism encountered in steel-reinforced elastomeric bearings. It limits total elastomeric shear strain due to axial load, rotation, and shear deformations, as well as distinguishes between static and cyclic component of shear strain. Method B provides for larger rotational capacity and is a versatile design procedure that allows for different combination of loadings.

The experience with accelerated bridge construction techniques on numerous bridge projects, such as the Medford Fast 14, shows that Method A previously provided a very narrow window of acceptable bearing design for a specific set of loadings. Furthermore, MassDOT studied 1,008 installed bearings to see just how much of additional rotation bearings actually see due to construction and fabrication and to be able to check the actual required bearing tolerances against the tolerances recommended by the *AASHTO LRFD*. This study found that these additional rotations greatly exceed the 0.005 radians currently prescribed in the *AASHTO LRFD* for construction tolerances, and that using a construction tolerance rotation of 0.03 radians would cover 90% of these actual rotations that were observed.

Presently, as per AASHTO M 251 specifications, bearings designed according to Method B require more extensive material and fabrication testing and therefore, it has historically been assumed that the required testing significantly increases their construction costs. Due to this fact and the apparent simplicity of Method A it was predominantly used by the state DOT's throughout the country. However, recent discussions with fabricators and testing laboratories indicate that this is not necessarily true due to the fact that most fabricators perform the testing required by AASHTO M 251 specification, as part of their normal QA/QC operations. Also, the independent testing laboratories report that the costs associated with the required testing are minor. Moreover, numerous tests performed as a part of the research for NCHRP Project 12-68 "Improved Rotational Limits of Elastomeric Bearings" show that more rigorous testing is needed for large bearings, and particularly thick ones, because they are more difficult to fabricate. Therefore, the bearing size, and not the design method employed, should be used as the criterion for more rigorous testing. Large bearings are defined as thicker than 8 in. or with a plan area of larger than 1000 in^2.

Therefore, based on the above the following shall apply to the design of the steel-reinforced elastomeric bridge bearings:

1. Method B procedure provides more rotational capacity than does Method A.
2. A construction tolerance of 0.03 radians shall be used instead of the 0.005 radians specified in the *AASHTO LRFD* since it more accurately reflects actual installed conditions.

3.5.7.4 Elastomer Material Properties. Based on Articles 14.7.5.2 and 14.7.6.2 of the *AASHTO LRFD*, the properties of elastomer material shall be as follows:

<u>Method B</u>: The elastomer shall be specified by Shear Modulus. The standard Shear Modulus to be specified for MassDOT bearings shall be taken as 0.160 ksi. The Shear Modulus for design purposes shall be taken as the least favorable value from the range of ±15% of the specified Shear Modulus, which for the Shear Modulus of 0.160 ksi is a range of 0.136 ksi (min.) and 0.184 ksi (max.).

<u>Method A</u>: The elastomer shall be specified by its Nominal Hardness on the Shore A scale. The Nominal Hardness of elastomer shall be 60 Durometer on the Shore A scale. In this case, the shear modulus for design purposes shall be taken as the least favorable value from the range for that hardness given in Table 14.7.6.2-1 of the *AASHTO LRFD*, which is between 0.130 ksi and 0.200 ksi.

3.5.7.5 Reinforcement. Steel laminates in steel reinforced elastomeric bearings shall conform to ASTM A1011 Grade 36 or higher and shall have a minimum thickness of 11 gage (0.1196"). The edges of all steel laminates shall be de-burred or otherwise rounded and ground smooth prior to being molded in the bearing to reduce the stress concentration in the elastomer at the critical location at the edge of the steel laminate. Tapered internal load plates shall conform to AASHTO M 270 Grade 36 or higher.

3.5.7.6 Design Procedure. The temperature to be used for the design of steel reinforced elastomeric bearings shall be the largest one-way thermal movement temperature specified in Paragraph 3.1.8.2

multiplied by the 1.2 *AASHTO LRFD* service limit state load factor. This design temperature shall be applied to all *AASHTO LRFD* bearing design equations.

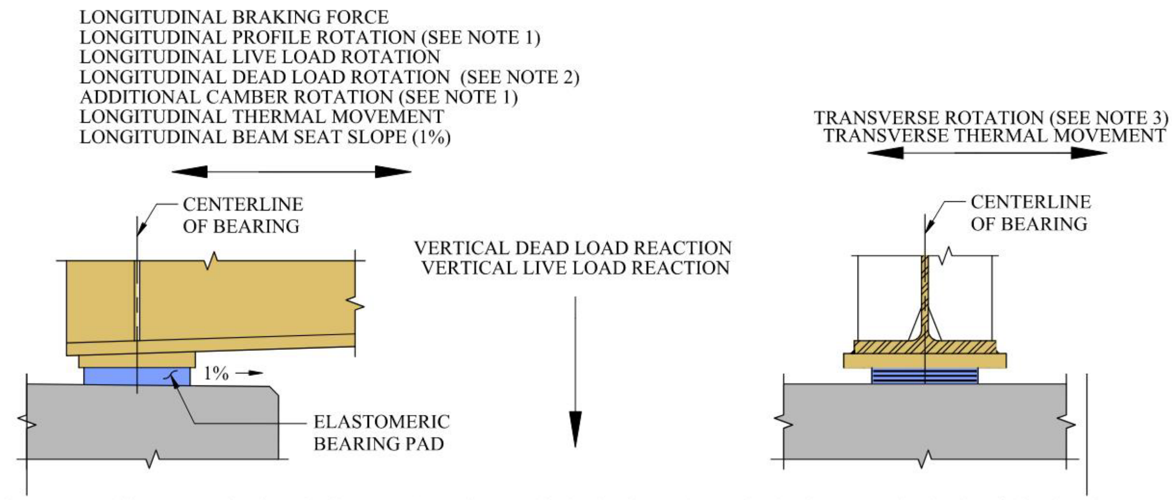
When using Method B, the basic equation for combined axial load, rotation, and shear at the service limit state (*AASHTO LRFD*, EQ 14.7.5.3.3-1) shall be limited to 4.75 instead of 5.0 in order to provide additional rotational capacity reserve. Unless otherwise noted, the resistance factor for bearings, ϕ, shall be taken as 1.0. Dynamic load allowance shall not be included. For bearings that have an internal load plate, only the elastomer layers and steel laminates below the load plate shall be used for the design of the bearing. The load plate and the upper cover layer are not considered part of the bearing and shall not be counted as elastomer layers or reinforcement and shall not be included as part of the bearing height for the bearing stability check.

The design rotation of bearing assemblies shall account for dead and live load rotations, rotation due to profile grade and beam seat slope, with an additional rotation of 0.03 radians, to account for uncertainties and construction tolerances. Note, MassDOT details require the entire beam seat to be sloped 1% towards the front face. The design rotation is assumed to be the vector sum of the longitudinal and transverse direction. Careful consideration shall be given to the effect of beveled sole plates (steel beam bridges) or internal beveled load plates (prestressed concrete beam bridges) and girder camber. For prestressed concrete beams, the net upward camber and associated end of beam rotations shall be calculated using the PCI “at erection” multipliers.

Method B requires the Designer to evaluate the bearing for both longitudinal and transverse effects. For example, the shear strain due to thermal movement is much different longitudinally (larger) than transversely (smaller). Likewise, the effect of vehicular braking force only affects the bearing in the longitudinal direction, however with accelerated bridge construction and on prestressed beams with heavy skews, especially box and NEXT beams, transverse rotations can be larger than longitudinal ones. The bearing needs to be designed using the resultant thermal movements and rotations. This will produce a conservative result as the maximum effect of the breaking force will not be coincident with the resultant movements or rotations. The 0.03 radians of additional rotation, for uncertainties and construction tolerance, shall be applied to the resultant rotation and not applied longitudinally and transversely.

Sole plates (steel beam bridges) or internal load plates (prestressed concrete beam bridges) may be beveled to account for the rotations due to profile grade. Ideally, properly beveled sole plates or internal load plates provide a level surface after the application of total dead load and after “at erection” camber (prestressed concrete beam bridges) has developed. If beveled sole plates or internal load plates are used, the design rotation for the elastomer due to profile grade should be neglected. When the required bevel of sole plates (steel beam bridges) or internal load plates (prestressed concrete beam bridges) is less than 1%, the required bevel (in radians) shall be included in the bearing design rotation and a flat sole plate (steel beam bridges) or no internal load plate (prestressed concrete beam bridges) shall be used.

The dead load design rotation of the elastomer should be neglected if the girder is cambered for dead loads (steel beam bridges). If the girder is not cambered the Designer shall account for the dead load rotation. In the case where a beveled internal load plate is used (prestressed concrete beam bridges), it shall be designed to accommodate the rotation due to profile grade, the dead load rotation, and the beam camber at erection. Figures 3.5.7-2 through 3.5.7-4 demonstrate the effects of girder cambering and a beveled sole plate (steel beam bridges) or internal beveled load plates (prestressed concrete beam bridges) on the rotation design of elastomeric bearings of a simple bridge (please note that the numbers shown are not specific to any bridge):



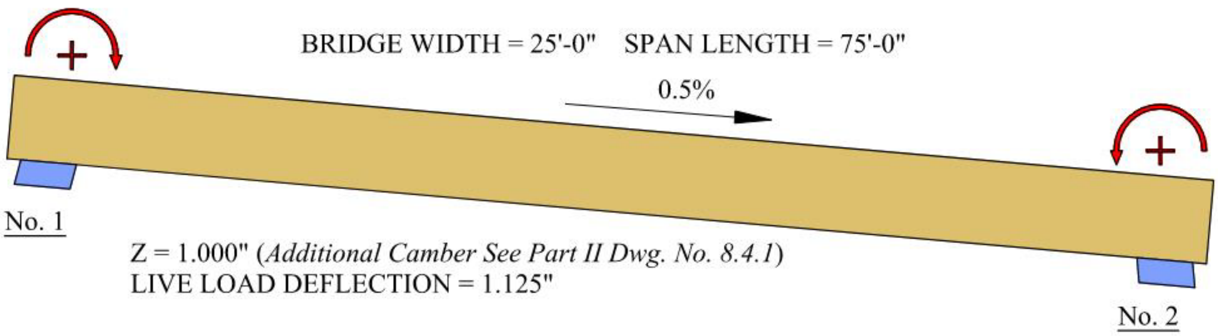
1. IF THE COMBINATION OF LONGITUDINAL PROFILE ROTATION AND ADDITIONAL CAMBER IS EQUAL TO OR GREATER THAN 1.0% THEN THESE ARE ADDRESSED THROUGH TAPERING OF THE SOLE PLATE AND IGNORED IN THE DESIGN.
2. DEAD LOAD ROTATION (CAMBER) IS TYPICALLY ZERO AS IT IS OFFSET BY THE BEAM CAMBER
3. TRANSVERSE ROTATION IS TYPICALLY ASSUMED TO BE ZERO DURING DESIGN, HOWEVER IT MAY BE PRESENT BASED UPON AS BUILT CONDITIONS IF A REVISED EVALUATION IS NECESSARY.

Figure 3.5.7-1: Bearing Design Forces and their Direction

It is desirable to have all bearings for a line of beams at a support to be the same. Therefore, the bearings should be designed for the first interior beam. The live load rotation should be calculated according to *AASHTO LRFD*, Article 2.5.2.6.2.

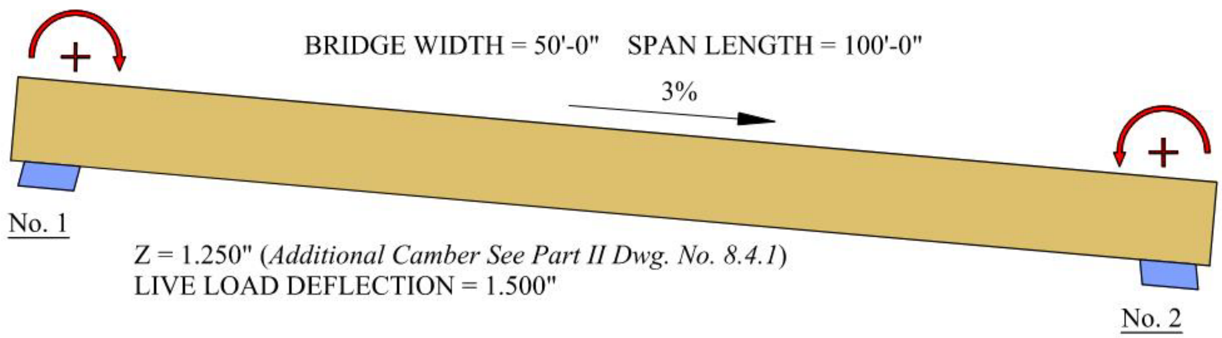
For exterior beams the Designer shall check the first interior bearing design using the exterior dead and live loads with the live load rotation used in the interior bearing design, however the *AASHTO LRFD* Equation 14.7.5.3.3-1 shall be limited to 5.0 instead of the 4.75 as noted above.

SAMPLE CALCULATION OF BEARING ROTATIONS AND THERMAL MOVEMENT FOR ELASTOMERIC BEARINGS



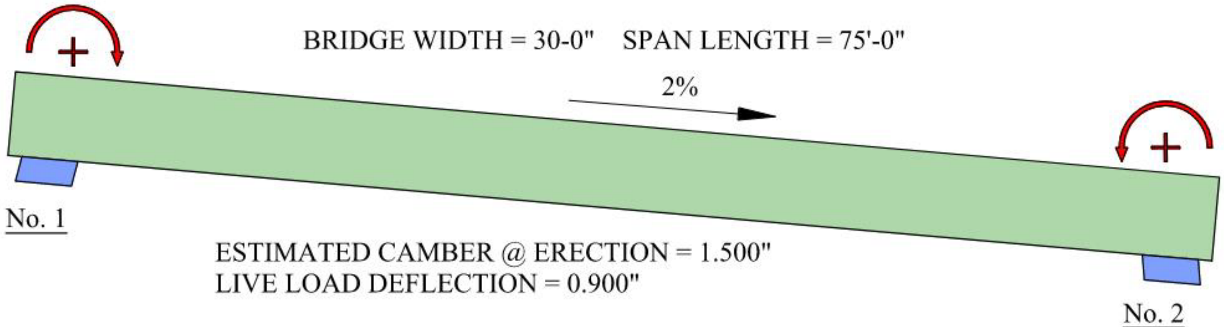
	No. 1	No. 2	
PROFILE GRADE ROTATION :	0.005 RAD	-0.005 RAD	*0.5% ROADWAY GRADE*
LONGITUDINAL DL ROTATION :	0.000 RAD	0.000 RAD	*BEAM CAMBERED FOR DEAD LOAD*
BEAM SEAT SLOPE 1% :	0.010 RAD	0.010 RAD	
ADDITIONAL CAMBER ROTATION :	-0.004 RAD	-0.004 RAD	*ADDITIONAL. CAMBER(Z)* $\frac{3.2x1.0}{80x12}$=*0.004 RAD*
SUBTOTAL :	0.011 RAD	0.001 RAD	
DESIGN LONG. ROTATION :	0.000 RAD	0.001 RAD	*USE BEVELED SOLE PLATE ON No. 1 NO BEVEL ON PLATE FOR No. 2*
LONGITUDINAL LL ROTATION :	0.004 RAD	0.004 RAD	← *LL ROTATION* $\frac{3.2x1.125}{100x12}$=*0.004 RAD*
TRANSVERSE ROTATION :	0.000 RAD	0.000 RAD	
$\sqrt{LONG^2 + TRANS^2}$	0.043 RAD	0.005 RAD	
	0.005 RAD		*USE LARGEST VALUE - DESIGN SAME BEARING FOR BOTH SUPPORTS*
UNCERTAINTIES & TOLERANCES :	0.030 RAD		
TOTAL DESIGN ROTATION :	0.035 RAD		← *DESIGN VALUE*
LONG. THERMAL MOVEMENT : SPAN = 75'-0" DT=100° (STEEL) :	0.351"		*1.2 x 0.5 x 0.0000065 x 75' x 12"/' x 70°= 0.208"*
TRANS. THERMAL MOVEMENT : WIDTH = 25'-0'' DT=70° (CONC) :	0.069"		*1.2 x 0.5 x 0.0000055 x 25' x 12"/' x 70°=0.069"*
$\sqrt{LONG^2 + TRANS^2}$	0.358"		← *DESIGN VALUE*

Figure 3.5.7-2: Steel Girder without Beveled Sole Plates



	No. 1	No. 2	
PROFILE GRADE ROTATION :	0.030 RAD	-0.030 RAD	*3% ROADWAY GRADE*
LONGITUDINAL DL ROTATION :	0.000 RAD	0.000 RAD	*BEAM CAMBERED FOR DEAD LOAD*
BEAM SEAT SLOPE 1% :	0.010 RAD	0.010 RAD	
ADDITIONAL CAMBER ROTATION :	-0.003 RAD	-0.003 RAD	*EST. CAMBER ROTATION DUE TO PRESTRESS* $\frac{3.2x1.25}{100x12}$*=0.003 RAD*
SUBTOTAL :	0.037 RAD	-0.023 RAD	*>1% THEREFORE USE INTERNAL BEVELED PLATES*
DESIGN LONG. ROTATION :	0.000 RAD	0.000 RAD	*USE INTERNAL BEVELED PLATE*
LONGITUDINAL LL ROTATION :	0.004 RAD		*LL ROTATION* $\frac{3.2x1.5}{100x12}$*=0.004 RAD*
TRANSVERSE ROTATION :	0.000 RAD		
$\sqrt{LONG^2 + TRANS^2}$	0.004 RAD		
UNCERTAINTIES & TOLERANCES :	0.030 RAD		
TOTAL DESIGN ROTATION :	0.034 RAD		*DESIGN VALUE*
LONGITUDINAL THERMAL MOVEMENT :			
SPAN = 100'-0" DT=100° (STEEL) :	0.468"		*1.2 x 0.5 x 0.0000065 x 100' x 12"/' x 100°= 0.351"*
TRANSVERSE THERMAL MOVEMENT :			
WIDTH = 50'-0'' DT=70° (CONC) :	0.139"		*1.2 x 0.5 x 0.0000055 x 25' x 12"/' x 70°=0.117"*
$\sqrt{LONG^2 + TRANS^2}$	0.488"		*DESIGN VALUE*

Figure 3.5.7-3: Steel Girder with Beveled Sole Plates



	No. 1	No. 2	
PROFILE GRADE ROTATION :	0.020 RAD	-0.020 RAD	*2% ROADWAY GRADE*
LONGITUDINAL DL ROTATION :	0.000 RAD	0.000 RAD	*ASSUME 0" PRESTRESS CAMBER IS > DL D*
BEAM SEAT SLOPE 1% :	0.010 RAD	0.010 RAD	
ADDITIONAL CAMBER ROTATION :	-0.005 RAD	-0.005 RAD	*EST. CAMBER ROTATION DUE TO PRESTRESS* $\frac{3.2x1.5}{75x12}$*=0.005 RAD*
SUBTOTAL :	0.025 RAD	-0.015 RAD	*>1% THEREFORE USE INTERNAL BEVELED PLATES*
DESIGN LONG. ROTATION :	0.000 RAD	0.000 RAD	*USE INTERNAL BEVELED PLATE*
LONGITUDINAL LL ROTATION :	0.003 RAD		*LL ROTATION* $\frac{3.2x0.90}{75x12}$*=0.003 RAD*
TRANSVERSE ROTATION :	0.000 RAD		
$\sqrt{LONG^2 + TRANS^2}$	0.003 RAD		
UNCERTAINTIES & TOLERANCES :	0.030 RAD		
TOTAL DESIGN ROTATION :	0.033 RAD		*DESIGN VALUE*
LONGITUDINAL THERMAL MOVEMENT : SPAN = 75'-0" DT=70° :	0.208"		*1.2 x 0.5 x 0.0000055 x 75' x 12"/' x 70°= 0.208"*
TRANSVERSE THERMAL MOVEMENT : WIDTH = 30'-0'' DT=70° :	0.083"		*1.2 x 0.5 x 0.0000055 x 30' x 12"/' x 70°=0.083"*
$\sqrt{LONG^2 + TRANS^2}$	0.224"		*DESIGN VALUE*

Figure 3.5.7-4: Prestressed Girder with Beveled Load Plates

For a simple span bridge the maximum rotation of the beam end can be calculated using normal stiffness methods. However, many beam design computer programs do not calculate the beam end rotation. An approximate beam end rotation can be determined based on maximum midspan deflection (please note that this is an exact solution only in the case when the beam is prismatic, and the beam deflection is parabolic):

- Calculate the maximum live load deflection at midspan Δ;
- Approximate end rotation in radians is equal to (3.2*Δ)/Span Length.

When determining the deflections and end rotations of continuous span bridges, the composite section properties shall be used for all segments of all beams. This includes the negative moment regions, where the transformed concrete slab should be used in place of the cracked section (beam and slab reinforcement).

3.5.7.7 Bearings shall also be designed for all longitudinal and lateral movements. Longitudinal translation due to dead load girder rotation about the neutral axis may need to be accounted for beams with large rotations or for deep beams. This translation should be added to the design longitudinal movement. The *AASHTO LRFD* outline requirements for calculation of thermal movement. The following are general guidelines that are intended to supplement the *AASHTO LRFD*:

<u>Standard Bridges:</u>

In this context a standard bridge is defined as a bridge that has the following geometric conditions:

1. Straight beams;
2. Skew angle ≤ 30 degrees;
3. Span length to width ratio greater than 2;
4. The bridge has 3 or less travel lanes.

The major contributor to thermal movements is the bridge deck. This portion of the bridge structure is exposed to the highest temperature extremes and is a continuous flat plate. A flat plate will expand and contract in two directions, and will not be significantly affected by other components of the superstructure below, i.e. beams, diaphragms and cross frames. For bridges that meet the general criteria listed above, the calculations for thermal movement can be based on the assumption that the bridge expands along its major axis, which is along the span length.

<u>Non-Standard Bridges:</u>

The treatment of non-standard bridges requires careful design and planning. A refined analysis may be required for non-standard bridges in order to determine the thermal movements, beam rotations (transverse and longitudinal), as well as the structural behavior of the system. The stiffness of substructure elements may also have an effect on the thermal movement at bearings. The following are general basic guidelines outlining the thermal movement behavior for non-standard bridges:

- Curved Girder Bridges:

It has been well documented that curved girder bridges do not expand and contract along the girder lines. The most often used approach is to design bearing devices to expand along a chord that runs from the point of zero movement (usually a fixed substructure element) to the bearing element under consideration.

- Large Skew Bridges:

The major axis of thermal movement on a highly skewed bridge is along the diagonal connecting the acute corners. The alignment of bearings and keeper assemblies should be parallel to this axis. The design of the bearings should also be based on thermal movement along this line.

- Bridges with small span-to-width ratios:

Bridges with widths that approach and sometimes exceed their lengths are subject to unusual thermal movements. A square bridge will expand equally in both directions, and bridges that are wider than they are long will expand more in the transverse direction than in the longitudinal direction. The design of bearing devices and keeper assemblies should take into account this movement.

- Wide bridges:

Bridges that are wider than three lanes will experience transverse thermal movements that can become excessive. Care should be taken along lines of bearings as to not to guide or fix all bearings along the line. Guides and keeper assemblies should be limited to the interior portions of the bridge that do not experience large transverse movements.

The Designer should specify on the Construction Drawings a range of temperatures for setting the bearings based on their design. Provisions should also be included for jacking the structure in order to reset the bearings if this range cannot be met during construction. A recommended temperature range is the average ambient temperature range for the bridge location plus or minus 10 °F. Larger values can be specified provided that the bearing is designed for the additional movement.

For continuous span bridges, bearings will see both minimum and maximum loads, depending on the location of the truck along the span of the bridge. In these situations, a bearing shall be designed and detailed for the maximum loading combination. The minimum loading combination shall be ignored in the bearing design.

Where anchor bolts are used to resist lateral forces, they shall be located outside the bearing pads and shall be designed for bending as well as shear. The sole plates shall also be checked for shear and bending.

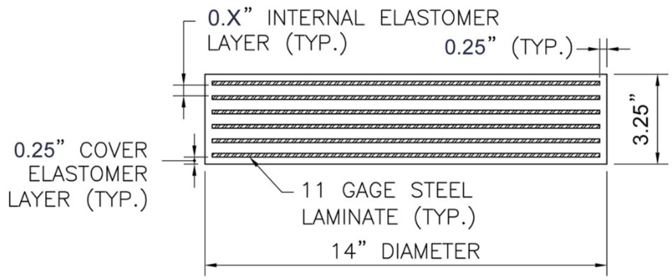
3.5.7.8 Detailing. Steel-reinforced elastomeric bearings shall be detailed on the Construction Drawings in accordance the standard bridge bearing details shown in Chapter 14 of Part II of this Bridge Manual. Bearing types not shown must receive prior approval from the State Bridge Engineer before being used in the design of a bridge project.

The thickness of the internal elastomer layer shall be calculated in decimal inches as shown in the example in Figure 3.5.7-5. This thickness shall be used in the design of the bearings as well as shown on the Construction Drawings. Using decimal inches is acceptable for elastomeric bearings since the bearing fabricators produce shop drawings that detail the bearing using decimal inches.

Tapered layers of elastomer in reinforced bearings are not permitted. If tapering of the bearing is necessary, it shall be accomplished as follows:

- For steel beams, provide an external tapered steel sole plate welded to the bottom flange.
- For concrete beams, use a tapered internal steel load plate and provide a cover layer of elastomer with constant thickness.

The minimum longitudinal slope of the bottom flange beyond which tapering of the bearing is required shall be equal to 1%. Refer to Paragraph 3.5.7.6 of this Section regarding situations with less than 1% bevels.



EXAMPLE FOR CALCULATING THE THICKNESS OF THE INTERNAL ELASTOMER LAYER FOR THE ABOVE BEARING:

1. SUBTRACT THICKNESS OF THE TOP AND BOTTOM COVER ELASTOMER LAYERS: 3.25 - 0.25 - 0.25 = 2.75 INCHES
2. SUBTRACT THE TOTAL ACTUAL THICKNESS OF ALL STEEL LAMINATES TO OBTAIN THE TOTAL THICKNESS OF ALL INTERNAL ELASTOMER LAYERS: 2.75 - (6 x 0.1196) = 2.0324 INCHES
3. DIVIDE THE TOTAL THICKNESS OF ALL INTERNAL ELASTOMER LAYERS BY THE TOTAL NUMBER OF THESE LAYERS (ROUND TO THREE DECIMAL PLACES): 2.0324 / 5 = 0.406 INCHES

USE THIS THICKNESS (0.406 INCHES) FOR DESIGNING THE BEARING AND FOR DETAILING THE BEARING ON THE PLANS AS FOLLOWS:

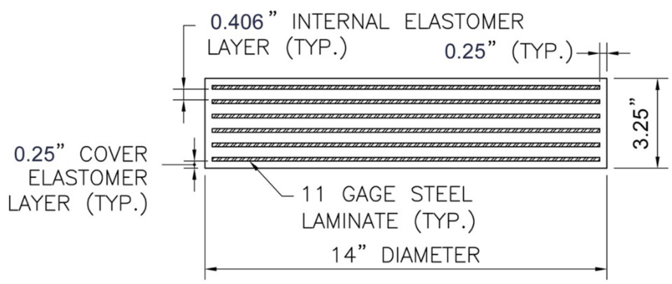


Figure 3.5.7-5: Calculation of Internal Elastomer Layer Thickness

3.5.7.9 Application. For adjacent concrete box and deck beam bridges with a span length of 50 feet or less, use rectangular plain (un-reinforced) elastomeric pads, 1" thick by 5" wide, detailed and placed as shown in Part II of this Bridge Manual.

For all other applications, circular steel-reinforced elastomeric bearings shall be used. The use of and detailing of rectangular steel reinforced elastomeric bearings must receive prior approval of the State Bridge Engineer.

3.5.7.10 Unfilled and lubricated PTFE (polytetrafluorethylene) sliding bearings shall only be used when a bearing with a low coefficient of friction is needed to minimize horizontal forces, i.e. thermal or seismic, on the substructure. Article 14.7.2, of the *AASHTO LRFD* shall be used to design this type of bearing. They shall be detailed on the Construction Drawings as shown in Part II of this Bridge Manual.

3.5.7.11 Anchor Bolts. When bearings with anchor bolts are used, in order to provide sufficient capacity to prevent failure of the concrete into which the anchor bolt is embedded, the embedment of each anchor bolt shall be designed conservatively to resist a pull out force equal to the sum of the design horizontal shear force applied to the anchor bolt plus any uplift force due to regularly applied loads that needs to be held down. The embedment depth of the anchor bolt shall be sized in conformance with Article 5.13 of the *AASHTO LRFD.*

3.5.7.12 High-Load Multi-Rotational Bearings. If the bridge reaction is too great for a standard elastomeric bridge bearing assembly to handle, then the Designer shall use a disc type high-load multi-rotational bearing instead. The advantage that a disc bearing has over a standard pot bearing is that the disc is exposed and can be readily inspected while the elastomeric component and the seals of a pot bearing are not.

3.5.8 Scuppers

3.5.8.1 An accurate determination of the need for scuppers on bridges as well as the design of deck drainage systems will be based on the latest edition of the Hydraulic Engineering Circular No. 21: *Design of Bridge Deck Drainage* (Publication No. FHWA SA-92-010).

3.5.8.2 The following may be used as a guide for estimating the need for scuppers and for locating them to properly drain the bridge superstructure:

1. On long bridges, scuppers should be placed about 350 feet on centers.
2. When the bridge is superelevated, scuppers are placed only on the low side.
3. On bridges, scuppers may be required when:
   a. The profile grade is less than 1%.
   b. The profile grade is such that ponding may occur on the roadway surface. An example would be a sag curve on the bridge.

The Designer shall investigate the highway drainage, which may include catch basins at the approaches to the structure.

3.5.8.3 When scuppers are needed, they shall generally be placed near a pier and on the upgrade side of a deck joint. Care shall be taken to ensure that scupper outlets will not result in run-off pouring or spraying onto either the superstructure beams or the piers.

3.5.8.4 Horizontal runs of drainpipes and 90° bends shall not be used. The minimum drainpipe diameter or width shall be 10”. The number of drainpipe alignment changes shall be minimized. Multiple alignment changes result in plugged scuppers that defeat the purpose of providing deck drainage. Cleanouts shall be accessible for maintenance purposes and shall be placed, in general, at every change in the alignment of the drainpipes. Typical details for scuppers and downspouts are shown in Part II of this Bridge Manual.

3.6 STEEL SUPERSTRUCTURES

3.6.1 General Guidelines

3.6.1.1 Coated AASHTO M 270 Grade 50 shall be the first choice of steel for all MassDOT steel bridges. Acceptable coatings and their application guidelines are given in Paragraph 3.6.1.2. Based on these guidelines, the use of uncoated weathering steel, AASHTO M 270 Grade 50W, will be limited exclusively to bridges over railroads alone with no adjacent roadways or bodies of water for the bridge to span over. Previously, AASHTO M 270 Grade 50W uncoated weathering steel was the primary option for all steel bridges constructed by MassDOT due to its perceived lower life cycle cost because it did not require periodic re-coating. However, evaluations of weathering steel bridges that have been in place for ten years or more have shown that the salt water spray kicked up by vehicles, even on secondary roads, contaminates the steel surface and prevents the protective patina from forming. This results in the continued corrosion of the steel, creates future maintenance problems, and negates the perceived life cycle cost benefits of weathering steel. Furthermore, bridge types where salt spray and dirt accumulation may be a concern (e.g., trusses or inclined-leg bridges) have also proven vulnerable to continued corrosion.

3.6.1.2 Acceptable steel coatings. Hot-dip galvanized steel will provide the best protection and should be specified wherever practical. However, the size of the galvanizing kettle typically limits the length of a beam that can be galvanized to about 60 feet (for a 36” deep beam). Due to the expense of preparing and painting galvanized steel as well as the care needed to ship painted beams to the field, galvanized beams should not typically be painted. If a painted beam is desired for aesthetic considerations, only the fascia beams shall be painted, while the interior beams can be left galvanized.

In addition to component dimensions, fabricated steel details can be very important to the galvanizing process and in some situations poor detailing can present a potential safety issue during galvanizing. The fabricated steel must allow for the easy flow of the molten zinc over and through it. Overlapping weld surfaces must be seal welded and drain and vent holes must be provided in the proper locations. It is the Designer’s responsibility to ensure this by evaluating all details, even MassDOT standards, for appropriateness to the galvanizing process. A resource which should be consulted is the American Galvanizing Association (AGA), either through the AGA publication, The Design of Products to be Hot-Dip Galvanized after Fabrication (available on their website), or with direct consultation with the association.

In order to reduce the need for field splices or to eliminate them altogether, for beams longer or larger than the limits specified above, metalizing should be specified as the coating method. Because the metalizing process results in a zinc coating that is somewhat porous, a sealer must be applied to all exposed surfaces to extend the life of the metalizing. Similar to galvanized beams, if a painted beam is desired for aesthetic considerations, only the fascia beams shall be painted with a three coat system, while the interior beams will be left metalized with a sealer. The type of thermal spray feedstock and the coating thickness shall conform to Table 3.6.1-1 below. The Designer shall specify on the Construction Drawings which zone is applicable.

Table 3.6.1-1: Application Requirements for Metalizing

ZONE	WIRE TYPE	THICKNESS (mils)**
Zone 1*	Zinc-Aluminum	6-10
Zone 2*	Zinc-Aluminum	8-12
Zone 3*	Zinc-Aluminum	10-14

***Zone 1** – Bridges in rural environments, not over waterways, and not over high speed state or interstate highways with potential for salt spray and heavy salt use and de-icing chemical use.

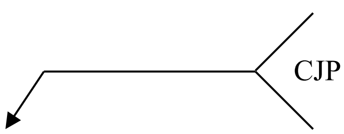
*__Zone 2__ – Bridges in urban environments, near industrial and manufacturing plants, power plants, or warehouses, over heavy road traffic, or over waterways.

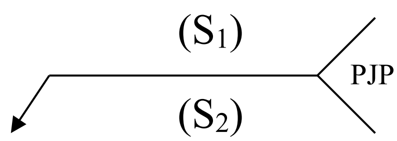
*__Zone 3__ – Bridges in marine environments, over or close to saltwater waterways, or over high-speed state or interstate highways with potential for salt spray and heavy salt use and de-icing chemical use.

** Mil thickness on faying surfaces shall meet the requirements of the slip certificate.

3.6.1.3 For all steel rolled beam and plate girder bridges, the ratio of the length of span to the overall depth of the beam (depth of the beam plus thickness of the design slab) shall preferably not be greater than 21. This ratio may be exceeded where due to clearance and profile requirements a shallower structure is required. However, for most conventional bridges, the span to depth ratio shall be no greater than 25 nor shall the span to depth ratio of the steel section alone be greater than 30. For continuous spans, the span length used in calculating this ratio shall be taken as the distance between dead load points of contraflexure.

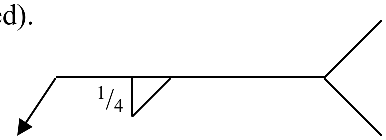
3.6.1.4 All welding and fabrication shall be in conformance with *the AASHTO/AWS Bridge Welding Code (AASHTO/AWS D1.5)*. The contract drawings shall clearly show the type of weld required. The drawings shall clearly distinguish between shop and field welds. For complete joint penetration (CJP) and partial penetration (PJP) groove welds, the drawings shall show the location and extent of the welds and, for the PJP welds, the required weld size. PJP groove welds shall not be allowed on primary members. These weld symbols shall be shown as follows:





Where S1 and S2 represent the effective throat size.

For fillet welds, the Construction drawings shall show the location, size and extent of the weld as shown below (1/4" weld indicated).



The fillet weld size to be specified shall be the larger of either the size as required by design or the minimum fillet weld size as given in Table 3.6.1-2 below.

Table 3.6.1-2: Minimum Fillet Weld Size

Base Metal Thickness of Thicker Part Joined (T)	Minimum Size of Fillet Weld
T ≤ ¾ inch	¼ inch [a, b]
T > ¾ inch	5/16 inch [a, b]

[a] Single-pass welds shall be used for the above welds

[b] Except that the weld size need not exceed the thickness of the thinner part joined.

3.6.1.5 All structural steel shall meet the requirements of AASHTO M 270. Primary members subject to net tensile stresses only, need to conform to the applicable Charpy V-Notch (CVN) Impact Test requirements of AASHTO M 270. A Primary Member is defined as any member making up the primary path that either the dead or live load takes from its point of application to its point of reaction onto the substructure, or in the case of steel bent piers, onto the foundation system. Refer to Table 6.6.2.1-1 of the *AASHTO LRFD*, for additional guidance. All primary members need to be identified on the plans. Secondary member steel shall conform to AASHTO M 270, excluding the CVN tests. ASTM A709 is similar to AASHTO M 270 and may be used in lieu of M 270 provided that the applicable CVN requirements for primary members are met.

3.6.1.6 Fracture critical members (FCM), or as they are now being called Nonredundant Steel Tension Members (NSTM), or member components, are primary members subject to net tension or tension components of bending members (including those that subject to the reversal of stress) whose failure may result in the collapse of the bridge. All FCM members and components, as defined in the *AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members,* shall be clearly designated on the contract drawings. All members and components designated as FCM are subject to the additional requirements of the Fracture Control Plan in the *AASHTO/AWS Bridge Welding Code*. Members and components not subject to net tensile stress under Strength I Limit State are not fracture critical.

If a bridge has fracture critical members, the Designer must also prepare and submit, as part of the design deliverables, a Fracture Critical Inspection Procedure prepared in accordance with the requirements of Subsection 3.13.2 of this Bridge Manual.

For bridges with a truss-floorbeam(-stringer) or girder-floorbeam(-stringer) floor systems, the floorbeams shall not be considered FCM if the spacing of the floorbeams is 12 feet or less and:

- The deck slab is designed to be continuous over the floorbeams with the main reinforcing placed parallel to the main trusses or girders.
- Or the stringers are placed on top of the floorbeams and at least every other stringer is continuous over a floorbeam.

In general, depending upon how they are connected to the primary members, secondary members, such as intermediate diaphragms on straight girder bridges, connection plates of diaphragms, transverse stiffeners, and lateral bracing should not be designated as fracture critical. Fracture critical requirements do not apply to temporary stages in construction.

3.6.1.7 The Designer shall locate and detail all field and transition splices. The location of these splices is dependent upon such factors as design criteria, available length of plates and members, ability to transport the members to the site, and erection and site limitations.

3.6.1.8 Where the Designer has an option to use either rolled beams without cover plates or welded plate girders for a structure, the Designer should consider rolled beams without cover plates. However, there may be situations where the welded plate girder is better suited to meet the project constraints, for example PBU's or profiles with large vertical curve middle ordinates. Some of the rolled beam sections may have limited production runs and may not be readily available. The Designer shall check to make sure that the specified sections are available. If the specified camber is excessive or if the structure has a radius less than 1200 feet, a welded plate girder design shall be considered instead of a rolled beam. Due to fabrication costs, rolled beams with cover plates should be considered as the last alternative.

3.6.2 Cover Plates

In situations where cover plates are necessary, the following provisions shall govern their use.

1. The minimum cover plate thickness shall be ½". For economy, it is preferable to use the same thickness cover plate on all similar size beams.
2. Bottom cover plates will be terminated not more than 2'-0" from the centerline of bearings or centerline of integral abutments, however the Designer must still check the fatigue stress range at the termination point.
3. Top cover plates, when used in the negative moment regions of continuous beams, shall extend beyond the theoretical end point by at least the terminal distance as defined in the *AASHTO LRFD*. Nonetheless, the actual termination point shall be determined by fatigue considerations.
4. The Designer shall design all cover plate to flange welds or shall verify the adequacy of the minimum weld sizes.

3.6.3 Welded Plate Girders

3.6.3.1 Minimum sizes for webs, flanges and welds, as well as detailing guidelines for plate girders, are given in Part II of this Bridge Manual.

3.6.3.2 The Designer shall first consider a web design that does not require the use of transverse stiffeners. If the required web thickness is excessive, a stiffened web will be considered; however, the spacing of the transverse stiffeners will be as large as possible. Cross frame connection plates can be used as stiffeners if they meet the *AASHTO LRFD* requirements for stiffener plates. For aesthetics, transverse stiffeners shall not be placed on the outside face of the exterior girders.

3.6.3.3 Longitudinal web stiffeners shall be avoided unless required by design to avoid excessively thick, transversely stiffened webs. Typically, longitudinal stiffeners should only be considered for very deep girders. If longitudinal stiffeners are used, they shall be placed on the opposite side of the web from the un-paired transverse stiffeners. Under no circumstances will longitudinal and transverse stiffeners be allowed to intersect. Shop splices of longitudinal web stiffeners shall be full penetration butt welds and shall be made before attachment to the web.

3.6.3.4 Flanges shall be sized as required by design, however for shipping and erection safety, the ratio of the shipping length to the width of the flanges shall be limited to 85 where practical according to Commentary C6.10.2.2 of the *AASHTO LRFD*, even at the expense of some additional steel.

3.6.3.5 The flange width may vary over the length of the girder; however constant width flanges are preferred. For longer spans where flange width transitions may be necessary, flange width transitions shall occur at the field splices. Top and bottom flanges need not be of the same width.

3.6.3.6 Straight welded plate girders less than 48" deep shall be considered rolled beams for the purposes of determine diaphragm depth according to Article 6.7.4.2 of the *AASHTO LRFD.*

3.6.4 Welded Box Girders

3.6.4.1 In general, the requirements for Welded Plate Girders contained in Subsection 3.6.3 shall apply to welded box girders.

3.6.4.2 The length of top flange used for the calculation of the length to width ratios for flanges contained in Paragraph 3.6.3.4 shall be based on the distance between internal shop-installed cross frames.

3.6.4.3 In general, the provisions for transverse web stiffeners contained in Paragraph 3.6.3.2 shall apply to box girders, except that all transverse stiffeners shall be placed in the interior of the box girder.

3.6.4.4 Longitudinal bottom flange stiffeners shall be avoided unless required by design to avoid excessively thick bottom flanges. Typically, longitudinal bottom flange stiffeners should only be considered for very wide flanges.

3.6.4.5 Box girder cross sections should be of a trapezoidal shape with webs sloped equally out from the bottom flange. Preferably, the minimum web depth shall be 6'-6" to allow for inspection access and maintenance activities inside the box girders. The minimum bottom flange width shall be 4'-0". Shorter web depths and narrower bottom flange widths may be used with the written permission of the State Bridge Engineer. In general, box girders placed on superelevated cross sections shall be rotated so that the top and bottom flanges are parallel to the deck cross slope.

3.6.4.6 Girder spacing shall be maximized in order to reduce the number of girders required, thereby reducing the costs of fabrication, shipping, erection, and future maintenance. Spacing of the top flanges in a bridge cross section shall be approximately equal, however, the spacing may be varied in accordance with *AASHTO LRFD* Article 6.11.2.3.

3.6.4.7 Utilities, including scupper drain pipes and street lighting conduit, shall not be placed inside the box girders. This restriction is imposed due to the fact that the interior of a box girder is a confined space with minimal lighting and placing any utilities only increases the hazards that inspectors or emergency responders would have to face.

3.6.4.8 At least two (2) inspection access hatches shall be provided in the bottom flange of box girders. These hatches shall be located and detailed such that Bridge Inspectors can gain access without the need for special equipment. The inspection access hatches are detailed in Chapter 8 of Part II of this Bridge Manual. If access hatches are accessible from the ground without ladders or equipment, the hatches shall be provided with an appropriate locking system to prevent unauthorized entry. Access openings shall be provided through all solid diaphragms. Stresses resulting from the introduction of access openings in steel members shall be investigated and kept within allowable limits.

3.6.4.9 The interior surfaces of box girders, including all structural steel components within the box girders (such as diaphragms, cross-frames, connection plates, etc.) shall be painted. The color of the interior paint shall be White (AMS Standard 595A Color Number 17925 of the Federal Standard 595B) in order to facilitate bridge inspection. So that that bridge inspectors can better orient themselves within the box girder, the distance from each box girder's West centerline of bearings, for bridges oriented generally west to east, or from the South centerline of bearings, for bridges oriented generally from south to north, shall be indicated in five (5) foot increments throughout the full length of each box girder. This distance shall be measured without interruption from the reference end of the box girder to the other end and shall be sequential over intermediate bearings and/or field splices within each box girder but shall not be carried over between separate box girders within the same girder line.

3.6.4.10 Top flange lateral bracing shall be provided to increase the torsional stiffness of individual box girder sections during fabrication, erection, and placement of the deck slab. Permanent internal lateral bracing shall be connected to the top flanges. Bracing members shall typically consist of equal leg angles or WT sections directly attached to the flange or attached to the flange via gusset plates. Filler plates shall be provided to accommodate the difference in elevation between connections.

The bracing shall be designed to resist the torsional forces across the top of the section and the forces due to the placement of the deck, satisfying the stress and slenderness requirements. The lateral bracing connections to the top flange shall be designed to transfer bracing forces. Pratt type bracing should be considered because of efficiency. X-bracing patterns should be avoided for economy. Forces due to any loads applied after the deck is cured shall not be considered in the connection of the bracing members or their connections. Allowable fatigue stress ranges shall not be exceeded where the gusset plates are connected to the flange.

3.6.4.11 The welds between the web and flanges shall be comprised of double fillet welds. If welding equipment cannot be placed within the box during fabrication, a complete penetration groove weld may be substituted with a backing bar on the inside and a reinforcing fillet weld on the outside. Backing bars shall be continuous or be made continuous prior to placement. Testing of welded splices in backup bars shall be treated similarly to flange splices.

3.6.5 Splices and Connections

3.6.5.1 Definitions. The term “Gusset Plate” shall only be used to refer to the plates that connect the primary members of a truss (i.e. diagonals, verticals and chords) at a panel point. All other plates used to connect secondary members to each other or to the primary members shall be called “Connection Plates”.

3.6.5.2 In general, all field connections shall be made with high strength bolts conforming to the requirements of ASTM F3125 Grade 325. All structural connections shall be designed as Slip-Critical connections. ASTM F3125 Grade 490 bolts shall not be used, except with written permission of the State Bridge Engineer.

3.6.5.3 Field splices in beams and girders, when necessary, shall generally be located as follows:

- Continuous Spans: Points of Dead Load contraflexure
- Simple Span: Quarter Point

3.6.5.4 Field splices shall generally be made using 7/8” ∅ high strength bolts. For large repetitive connections, the use of larger bolts shall be evaluated if a significant number of bolts could be saved. All bolts used in a splice shall be of the same diameter. Filler plates shall not be less than ⅛” thick and shall extend to the limits of the splice plate. Field splices of flanges and webs shall not be offset.

3.6.5.5 Transverse stiffeners will be located as specified in Part II of this Bridge Manual so that they do not coincide with the splice plates. If stiffeners in the area of a bolted splice are unavoidable, bolted steel angles shall be used as stiffeners instead of plates welded to the splice plates.

3.6.5.6 As welded flange splices are costly, a savings of approximately 1300 pounds of steel should be realized in order to justify the cost of the flange splice. Due to the cost of making a full penetration welded flange splice, the number of changes to the flange thickness should be kept to a minimum but in no instance shall exceed 3 different sizes between bolted splices. When a girder flange is butt spliced, the thinner segment shall not be less than one-half the thickness of the adjoining segment.

3.6.6 Prefabricated Bridge Units (PBUs)

3.6.6.1 General. The framing plan concepts presented in Chapter 6 of Part III of this Bridge Manual do not require diaphragms or cross frames between PBU units. This is based upon the primary function of these members which are to brace the beams during the placement of the concrete deck. However, since the deck has already been placed and cured these diaphragms and cross frames are no longer necessary. This has no effect on the live load distribution factors calculated from AASHTO LFRD Table 4.6.2.2.2b-1, for interior beams, or Table 4.6.2.2.2d-1, for exterior beams, as these equations were developed ignoring all intermediate diaphragms or cross frames (refer to *AASHTO LRFD C4.6.2.2.b*). However, end diaphragms or cross frames are necessary. The standard MassDOT end diaphragm concrete encasement, with or without steel diaphragms or cross frames, meets this requirement.

Special attention needs to be paid to PBU's with standard utility bays. The top and bottom utility supports are insufficient to brace the beam during the casting of the deck to prevent the beams from shifting out of alignment. Chapter 6 of Part III of this Bridge Manual provides additional guidance and details.

Regarding pile cap analogy, a study was conducted analyzing typical PBU configurations for various span lengths. It was found that the actual distribution of dead and live loads for the exterior beams resulted in less load applied to the exterior beam than if pile cap analogy or lever rule was used. Therefore, the provisions of Subsection 3.5.3 above shall be followed.

3.6.6.2 Live Load Continuity for PBUs. The procedures outlined here are for multi-span bridges composed of PBUs with continuity diaphragms cast between the girder ends at the pier. These superstructures shall be designed as simple spans for all non-composite dead loads and as continuous girders for all composite dead loads and live loads applied after continuity is established and shall be detailed as per Part III of this Bridge Manual.

3.6.6.3 The connection between PBUs at the continuity diaphragm (i.e. closure pour) shall be designed for only negative moments. The flexural tension is resisted by the longitudinal reinforcement in the concrete deck and the designer shall ensure that adequate detailing is provided. Flexural compression is resisted by the shear connectors welded to the bottom flange and web of the girders being connected. The design is based on the couple that exists between the centroid of the longitudinal deck reinforcement and the centroid of the shear connectors. The design shall include checks for the Strength, Fatigue and Service II Limit States.

3.7 PRESTRESSED CONCRETE SUPERSTRUCTURES

3.7.1 Standard Beam Sections

3.7.1.1 Standard prestressed concrete deck, box, NEBT, NEDBT, NEXT, or NEXT D beam sections as detailed in Part II and III of this Bridge Manual shall be used to construct prestressed concrete bridge superstructures. Other sections may be used where the situation precludes the use of standard sections and prior written approval has been obtained from the State Bridge Engineer, or where so permitted by this Bridge Manual.

3.7.1.2 The standard beam sections were developed in conjunction with PCI Northeast and meet the fabrication tolerances and practices of most regional Fabricators. If a particular design requires that major alterations be made to the standard details, such as the placement of strands in locations other than those shown or different reinforcing details, it will be the Designer's responsibility to ensure that the design can be fabricated by a majority of area Fabricators.

3.7.1.3 In adjacent prestressed beam, NEXT F, NEXT D, and NEDBT superstructures, the beams should be placed to follow the roadway cross slope as much as is practical. On bridges with a Utility Bay under the sidewalk, the sidewalk beam need not be placed to follow the cross slope, unless a deeper sidewalk depth is required over this beam for railing/traffic barrier attachments. For NEBT or spread box beam bridges, the beams shall be placed plumb and a deck haunch deep enough to accommodate the drop of deck across the width of the beam flange shall be provided.

3.7.2 Materials and Fabrication

3.7.2.1 Concrete Strength. Designs of prestressed concrete Deck and Box beams, as well as NEBT and NEDBT beams, shall be based on a concrete compressive strength (f'_c) of 6500 psi. To avoid going to a deeper beam and if required by design parameters, a concrete compressive strength of 8000 psi may be used. Use of concrete strengths other than these two standard mixes is discouraged, since this will require the Fabricators to prepare a special mix design and receive approval for it by MassDOT prior to fabrication, which will delay the start of fabrication and add to the cost of the beams. For a design concrete compressive strength (f'_c) of 6500 psi, the concrete compressive strength at release (f'_{ci}) shall generally be taken as 4500 psi. Higher concrete release strengths, up to 0.8 f'_c, may be used only if required by design in order to avoid going to a deeper beam. Concrete release strengths greater than 0.8 f'_c shall not be used.

Designs of NEXT F and NEXT D beams shall be based on a concrete strength (f'_c) of 8000 psi. The concrete compressive strength at release (f'_{ci}) shall be taken as 6000 psi.

3.7.2.2 Prestressing Strands. Only Low Relaxation strands meeting the requirements of AASHTO M 203 shall be used as required by MassDOT Specifications. Strands shall be 0.6” in diameter and shall not be epoxy coated. Beams shall be fabricated with the prestressing strand layout as shown on the Construction Drawings. The concrete gross section shall be used to compute section properties (the transformed area of the prestressing strands shall not be used for this purpose).

For ease of fabrication, Fabricators prefer to use straight, debonded strands over draped strands in order to reduce the tensile stresses at the ends of Box beams, NEBT beams, and NEDBT beams. The draping of strands shall be used only if debonding alone, due to the limitations imposed on de-bonding as specified below, will still result in unacceptably high tensile stresses. In this situation, mixing draped and debonded strands in a beam will be permitted. For Deck beams, NEXT F and D beams, due to their detailing, draped strands cannot be used.

Where draped strands are used, the total hold down force of all draped strands for each beam shall not exceed 75% of the total beam weight.

3.7.2.3 Reinforcing Steel. All non-prestressed reinforcement shall be epoxy coated Grade 60 reinforcing steel. It is the Designer's responsibility to detail the beams so that all reinforcement is embedded, developed or lapped as required. In the case of adjacent deck or box beams, the size of the void may need to be reduced (or eliminated for deck beams only) to allow for proper bar development of barrier reinforcement, as noted in Part II of this Bridge Manual.

3.7.2.4 Utility Supports. The steel for all utility supports shall conform to AASHTO M 270 Grade 36 or Grade 50 and shall be galvanized. All inserts for the attachment of utilities will be cast into the beam at the time of its fabrication. Under no circumstances shall expansion type anchors be allowed. Inserts that are being provided for a future utility installation shall be furnished with a plastic plug that is the same color as the concrete. Once the beam is cast, the drilling of holes for attachments will not be permitted.

3.7.3 General Design Requirements

3.7.3.1 All prestressed beams shall be designed for all applicable limit states and for all loading conditions according to the *AASHTO LRFD,* except where modified and/or amended by this section.

3.7.3.2 All prestressed beams shall be designed to have no more than $0.0948\sqrt{f'_c}$ ksi tension in the pre-compressed tensile zone under Service III limit state after all losses have occurred. If the only way to reduce these tensile stresses is to go to the next larger beam size and the depth of structure is critical, tensile stresses up to a maximum of $0.19\sqrt{f'_c}$ ksi will be permitted. In this case a live load factor of 1.0 for the Service III Load Combination should be used.

3.7.3.3 The top and bottom #4 U-shaped bars shall be lapped to form the transverse stirrups and shall be designed so that their vertical legs satisfy shear reinforcement requirements of Article 5.7 of the *AASHTO LRFD*.

3.7.3.4 Horizontal shear reinforcement for making decks composite with prestressed beams shall be designed in accordance with the *AASHTO LRFD* and placed together with the transverse stirrups, as noted in Part II of this Bridge Manual.

3.7.3.5 End transverse stirrups and end vertical stirrups shall be provided as shown in Part II of this Bridge Manual and shall be designed to satisfy the *AASHTO LRFD* for splitting resistance of pretensioned anchorage zones. These bars should be placed within a distance h/4, as defined in the referenced Article depending on the type of the beam, from the end of a beam and should be either #4 or #5 bars. If using #5 bars, the lap length and embedment length shall be adjusted as needed.

3.7.4 Continuity Design for Prestressed Concrete Beam Bridges

3.7.4.1 General. The procedures outlined here are for multi-span bridges composed of prestressed spread and adjacent deck and box beams, NEBT beams, NEXT F beams, NEDBT beams and NEXT D beams with continuity diaphragms cast between ends of girders at piers that are made continuous without the use of post-tensioning. These superstructures shall be designed per Article 5.12.3.3 of the *AASHTO LRFD* as simple span beams for all non-composite dead loads and as continuous beams for all composite dead loads and live loads applied after continuity is established and shall be detailed as per Part II of this Bridge Manual.

3.7.4.2 Design and Detailing. The connection between prestressed girders at a continuity diaphragm (i.e. the closure pour) shall be considered partially effective and shall be designed for all effects that cause moment at the connection, including restraint moments from time-dependent or other deformations. These restraint moments shall not be included in any combination when the effect of the restraint moment will reduce the total design moment.

Both a negative and a positive moment connection are required for continuity diaphragms. The negative moment connection shall be designed as per Article 5.12.3.3 of the *AASHTO LRFD*. To establish a positive moment connection, MassDOT's practice is to extend prestressing strands beyond the end of the girder and anchor them into the continuity diaphragm. Only fully bonded strands in the bottom row shall be used for this purpose. Design for this positive moment connection is not necessary when using the standard full depth diaphragm detail, which is different from the closure detail between beams assumed in the AASHTO provisions.

Multi-span bridges constructed of prestressed spread and adjacent deck and box beams, NEBT beams and NEXT F beams shall be designed compositely with the cast-in-place deck slab with the continuity reinforcement placed in the deck slab. Gross composite girder section properties, ignoring any deck cracking, may be used for analysis. The design compressive strength (f'_c) of the prestressed girder as well as its bottom flange width shall be used to calculate flexural resistance of the continuity reinforcement used for the negative moment connection over a pier.

3.7.5 Design of NEBT Beams Post-Tensioned for Continuity

3.7.5.1 NEBT beams were developed so that they could be post-tensioned to be fully continuous (i.e. continuous for all dead loads and live loads) or to splice several segments together to form continuous beams longer than what could be achieved by using the simple span procedures in Subsection 3.7.4. Designers should consider the benefits of post-tensioned continuity when evaluating these beam superstructures.

3.7.5.2 At present, this Bridge Manual does not contain details for the post-tensioning NEBT beams. For more information and for details, Designers shall refer to PCI Northeast Report PCINER-01-PTDG, which can be downloaded from the URL: www.pcine.org.

3.7.5.3 NEBT beams post-tensioned for continuity or spliced to form longer continuous beams shall be designed in accordance with Article 5.12.3.4 of the *AASHTO LRFD*.

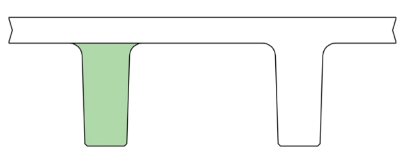
3.7.5.4 Since the post-tensioning ducts need to be filled with grout after the tendons have been stressed, the grout to be used, the proper procedures to be followed, and venting of the ducts to ensure that they will be filed without voids are critical. The Designer shall confer with the Bridge Section about all these requirements.

3.7.6 Longitudinal Stiffness Parameter, Kg, for NEXTD & NEDBT Beams

3.7.6.1 General. The Live Load Distribution Factors provided in the *AASHTO LRFD*, reference a Longitudinal Stiffness Parameter, Kg. In the calculation of Kg, *AASHTO LRFD* assume a concrete deck made composite with a basic beam and uses the moment of inertia and area of the beam along with the distance between the center of gravity of the basic beam and the deck. In the case of NEXT D beams and NEDBT beams, the top flange and the rest of the beam have been cast monolithically and thus, adjustments to the AASHTO calculations are required.

Therefore, the Kg shall be calculated assuming the basic beam is the portion of the prestressed unit below the top flange, see Figures 3.7.6-1 and 3.7.6-2 below. The Kg values for both NEXT D and NEDBT standard beams have been calculated and provided in the following tables. These values should be used for design purposes.

BEAM	Kg (in^4)
NEXT 28D	62100
NEXT 32D	98700
NEXT 36D	147300
NEXT 40D	209700



PORTION OF NEXT D
USED IN Kg CALCULATION.

Figure 3.7.6-1: Kg Values for NEXT Beams

BEAM	Kg (in^4)
NEDBT 40	366600
NEDBT 48	607000
NEDBT 56	922100
NEDBT 64	1319300
NEDBT 72	1805700
NEDBT 80	2388700

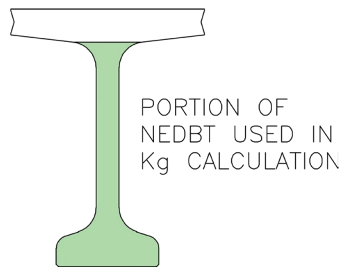


Figure 3.7.6-2: Kg Values for NEDBT Beams

3.8 ADJACENT PRESTRESSED CONCRETE DECK AND BOX BEAM BRIDGES

3.8.1 General Requirements

3.8.1.1 Background. In the past, adjacent prestressed concrete deck and box beam bridges were built with just a membrane and HMA wearing surface placed directly on top of the beams of these bridges. Over the years, this type of construction has suffered from shear keys that deteriorated and leaked, leading to the deterioration of the beams themselves. This was especially so for bridges with high ADT's and ADTT's. In the early 2000's MassDOT restricted this type of adjacent deck and box beam construction to bridges with ADT's of less than 5,000. Since the 2005 Bridge Manual, MassDOT has required the use of a 5" minimum thickness cast in place reinforced concrete deck slab on top of these beam systems and removed the ADT restriction. The purpose of this deck was twofold. First, it would reduce the fatigue on the shear keys because the deck would help distribute the live load between the beams. Second, it would be a sacrificial element that could be replaced without replacing the adjacent beam system itself.

3.8.1.2 This type of bridge is considered to be Typical Cross Section (f) as shown in the *AASHTO LRFD*, Table 4.6.2.2.1-1. The beams in the adjacent beam systems shall be designed to be composite with this deck slab by casting dowels into the beams that have been designed for horizontal shear as specified in Article 5.7.4 of the *AASHTO LRFD*.

3.8.1.3 Utility Bay under Sidewalk and Special Sidewalk Beam. The special sidewalk beam as defined in Figure 3.8.1-1 may be either a standard deck or box beam section or a rectangular solid prestressed beam. NEBT beams shall not be used for this application. If the sidewalk is wide enough to accommodate two or more standard deck or box beam sections as special sidewalk beams, provide longitudinal joints and transverse ties between them as for regular adjacent beams and distribute Superimposed Dead Loads, Pedestrian Loads and Live Loads to each sidewalk beam using the procedures outlined in Subsection 3.8.2.

The special sidewalk beam(s) shall be designed to be composite with the sidewalk slab. The dowels cast into the beam(s) shall be designed for horizontal shear as specified in the *AASHTO LRFD*. The effective width of the slab shall extend to mid-bay.

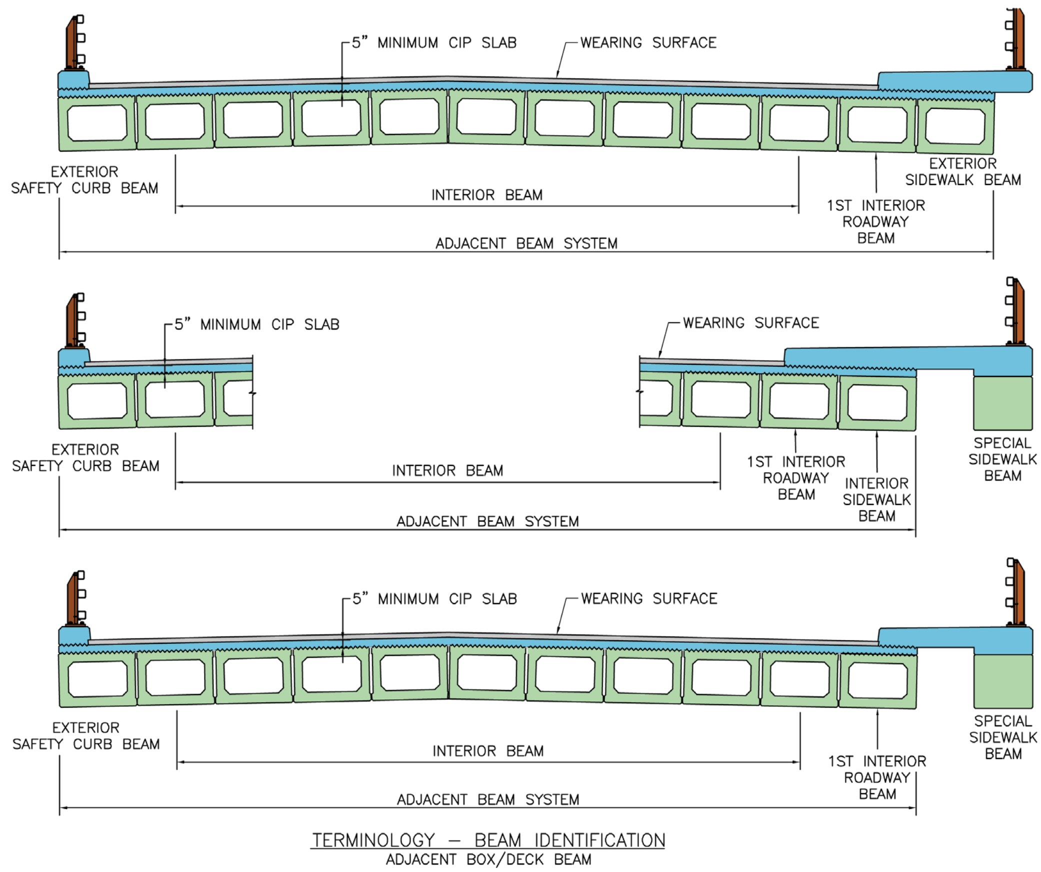


Figure 3.8.1-1: Adjacent Box/Deck Beam Terminology for Beam Identification

3.8.2 Distribution of Loads to Adjacent Prestressed Concrete Beam Bridges

3.8.2.1 The distribution of loads to these bridges shall be in accordance with Subsection 3.5.3, except as modified by this Subsection. The HL-93 live load shall be distributed in accordance with the *AASHTO LRFD* distribution procedures for these types of bridges assuming that the beams are non-composite, however, the composite section properties shall be used to design the beams and to check stresses.

3.8.2.2 Superimposed Dead Load Distribution (DC2) and Pedestrian Load (PL). For adjacent beam systems without a special sidewalk beam, see Figure 3.8.2-1 below. In using the pile cap analogy model to distribute these loads to the exterior sidewalk beam, the 1st interior roadway beam and the exterior safety curb beam, the theoretical point of support provided by a beam shall be located at its centerline. When using the pile cap analogy, the relative individual stiffness of the beams shall be ignored, i.e. A = 1.0 for all beams, since the unequal spacing of the pile cap points of supports approximates this effect. The Pile Cap analogy as referenced in the *AASHTO LRFD* Article 4.6.2.2.2d is used for distributing the HL-93 Live Load to exterior beams in beam-slab bridge cross sections, and so, is not used for live load distribution to bridges with a Typical Cross Section (f). However, MassDOT does not consider that an equal distribution of the sidewalk slab or safety curb/barrier loads accurately models the actual distribution of these loads to the beams in the adjacent beam system. Therefore, MassDOT considers the pile cap analogy to be a better dead load distribution model and applies it here in lieu of a simplistic 60% distribution as was used in the past.

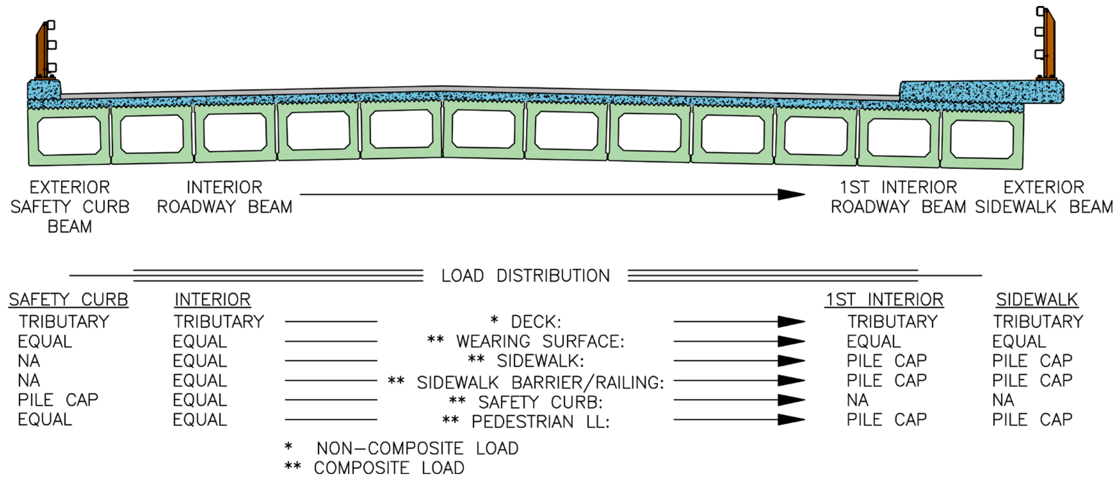


Figure 3.8.2-1: Load Distribution to Adjacent Box/Deck Beams

For adjacent beam systems with a special sidewalk beam, see Figure 3.8.2-2 below. The sidewalk slab is placed non-compositely and shall be distributed by tributary area as a non-composite dead load to the special sidewalk beam while the full sidewalk railing/barrier load and tributary area of the pedestrian load shall be applied as composite loads. The wearing surface dead load that is applied to the adjacent beam system shall not be applied to the special sidewalk beam. Similarly, the sidewalk railing/barrier load is fully applied to the special sidewalk beam and is not distributed to beams of the adjacent beam system. The rationale behind this distribution is that the special sidewalk beam, because it is not connected to the rest of the adjacent beam system with shear keys, it will deflect semi-independently of the adjacent beam system. Therefore, this load distribution is simple but conservative.

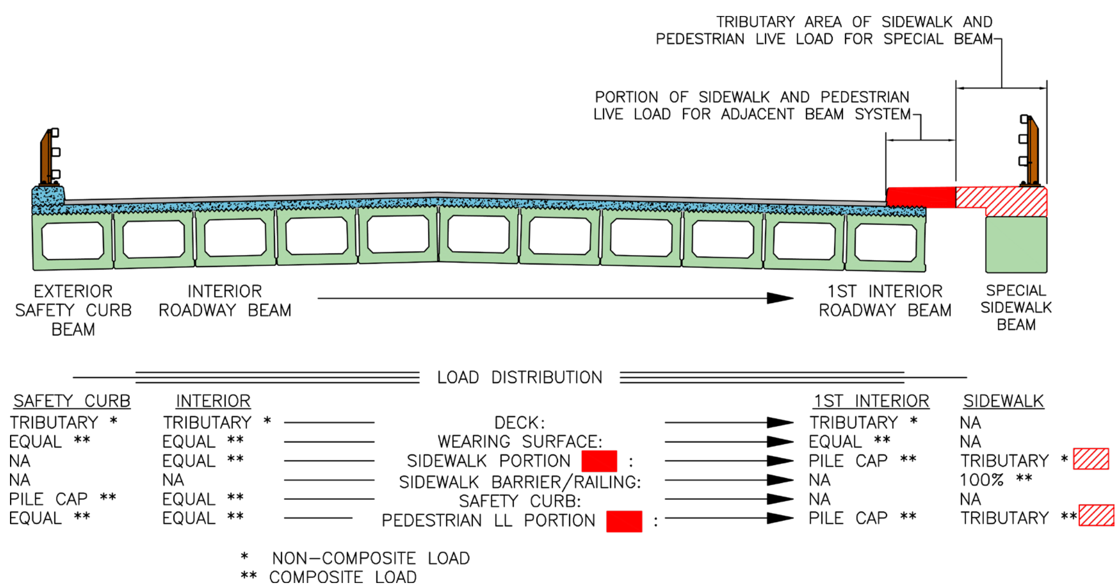


Figure 3.8.2-2: Distribution of Loads for Utility Bay under the Sidewalk with Special Sidewalk Beam

3.8.2.3 For bridges with a configuration similar to Figure 3.8.2-3 below, with or without utilities, Paragraph 3.5.3.11, Case II (Truck on Sidewalk) shall be ignored since the AASHTO live load distribution factors account for the effect of the truck on the exterior sidewalk beam.

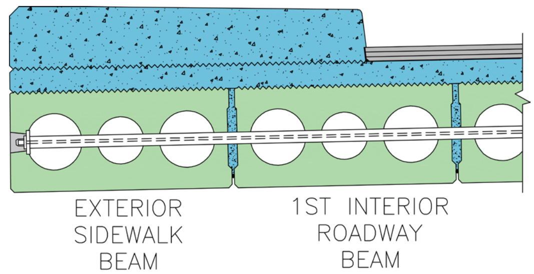


Figure 3.8.2-3: Adjacent Beam Bridge with no Utility Bay

3.8.2.4 For bridges with a configuration similar to Figure 3.8.2-4 below, with or without utilities, Paragraph 3.5.3.11 Case I shall be modified by considering the 1st Interior Roadway Beam to be an exterior beam for distribution of live load purposes and the Special Sidewalk Beam shall not be considered in this case. Case II shall be modified by distributing the wheel lines as follows: if the wheel line is located anywhere over the special sidewalk beam, apply 100% of the wheel line load to this beam; if the wheel line is located over the utility bay (between the beams), distribute the wheel line load to the special sidewalk beam using the lever rule and assuming that the hinges are placed at the edge of the beams supporting the sidewalk slab span. Depending on the width of the sidewalk, use one or both wheel lines. The load carrying effect of the 1st Interior Roadway Beam shall not be used to reduce the live load effect on the special sidewalk beam. If there is more than one Special Sidewalk Beam, the wheel load effects that are to be distributed to the special sidewalk beams using the above procedures shall be divided equally among all of these beams.

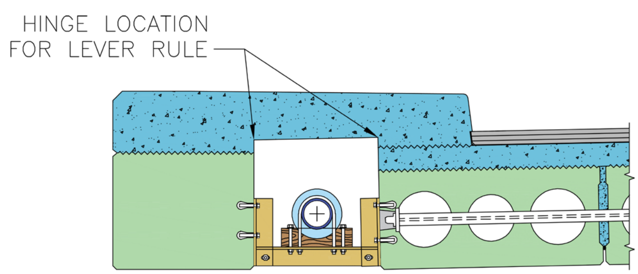


Figure 3.8.2-4: Adjacent Beam Bridge with Utility Bay and Special Beam

3.8.2.5 Effect of Different Moments of Inertia on the Distribution of Loads to all beams in the adjacent beam system. If beams of different Moments of Inertia are used together in an adjacent beam superstructure, the Superimposed Dead Loads and Pedestrian Load (if any) shall be distributed to each beam in the adjacent beam system in proportion to its Moment of Inertia according to the following formula:

$$L.D.F._k = \frac{I_k}{\sum_{i=1}^{n} I_i}$$

In the formula above, L.D.F.$_k$ is the load distribution factor for the k^{th} beam, I_k is the Moment of Inertia of the k^{th} beam, and I_1 ... I_n are the Moments of Inertia of each beam in the adjacent beam system.

The live load distribution factors as computed per Section 4 of the *AASHTO LRFD* are a function of the beam's width and Moment of Inertia, and therefore, no further distribution of Live Load is required.

3.8.2.6 Design of the Sidewalk Slab over a Utility Bay. The sidewalk slab shall be designed for the differential deflection between the adjacent roadway beam system and the special sidewalk beam(s).

STEP 1: Calculate the deflection of the adjacent roadway beam system by placing the factored HL-93 loads in each of the actual travel lanes (not the AASHTO design lanes) and assuming that all adjacent beams act and deflect together.

STEP 2: Calculate the equivalent uniformly distributed load (per foot of beam) that would cause the same deflection in the special sidewalk beam as calculated in Step 1. Use the composite section properties. If there are two or more special sidewalk beams, calculate the load that would deflect all special sidewalk beams at once. Since this load was derived by using factored HL-93 loads, it is considered to be a factored load.

STEP 3: The sidewalk slab shall be considered a cantilever beam with a length equal to the clear width of the utility bay. The factored design load shall be the uniform load calculated in Step 2 and applied at the free end of the cantilever. Assume the section to be singly reinforced and use the smallest d dimension. Design the required steel area using factored resistance and provide it for both top and bottom transverse slab reinforcement. Spacing of these bars should be at a multiple of the sidewalk dowels of the first interior roadway beam.

STEP 4: Design the sidewalk dowels that connect the sidewalk slab to the first interior roadway beam to concurrently resist the design moment used to design the sidewalk slab in Step 3 and the equivalent factored load calculated in Step 2. Factored resistances shall be used.

If excessive steel areas are required, consideration should be given first to increasing the depth of the sidewalk slab and, second, by providing intermediate diaphragms. The intermediate diaphragms need only be designed for the load in excess of the slab capacity.

3.8.3 Utilities on Adjacent Prestressed Concrete Deck and Box Beam Bridges

3.8.3.1 General. Utilities shall be located as shown in Chapter 10 of Part II of this Bridge Manual. Preference shall be given to locating the utilities in the utility bay under the sidewalk wherever possible. Under no circumstances shall utilities be located inside the beams within the void.

The utility supports shown in Part II of this Bridge Manual represent acceptable configurations. Where utility support member, bolt and insert sizes are provided, these supports may be used up to the limits shown without further design. These supports may have to be altered depending on the utility. If an increase in the side clearance of the utility bay is required, the L4x4x½ attached to the side of the beam may be replaced by an attachment using a section of WT. In these cases, the Designer is responsible for the design of the utility supports. In all cases, the utility supports must be adequately detailed on the Construction Drawings.

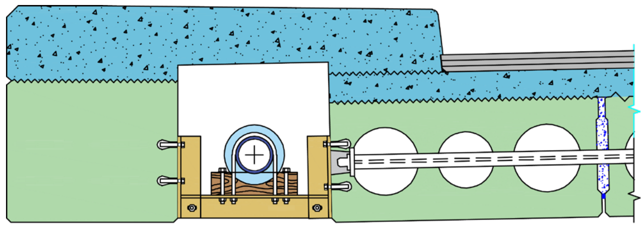


Figure 3.8.3-1: Utility under the Sidewalk

For utilities supported similar to Figure 3.8.3-1, the utility loads shall be considered non-composite and generally be assumed to be equally distributed to the beams that support them on either side of the utility bay.

For utilities supported similar to Figure 3.8.3-2, these loads shall be considered composite and be distributed using the Pile Cap analogy. Whenever a utility is attached to the exterior of an adjacent beam bridge, the torsional effect of such an attachment may cause unequal reactions at the bearings. This effect may be compounded by additional eccentric loads, such as either a sidewalk overhang or a safety curb with a railing/barrier, which does not extend over to the second interior beam. To help equalize the reactions at the bearings, consideration should be given to increasing the number of transverse ties.

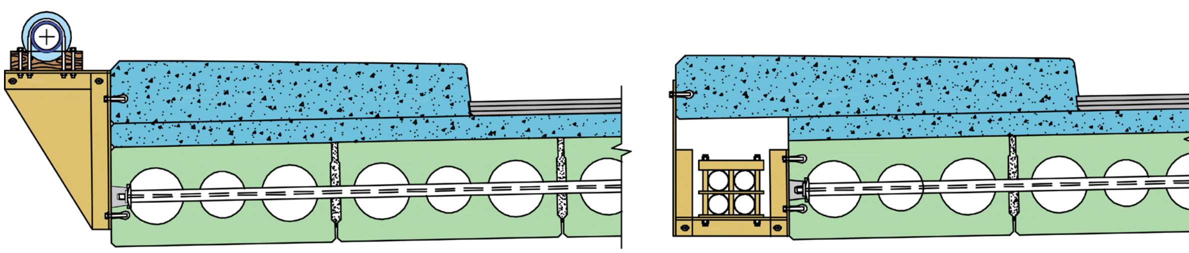


Figure 3.8.3-2: Utility Installed on the Exterior of an Adjacent Beam Bridge

3.9 PRECAST CONCRETE FULL DEPTH DECK PANELS (FDDP)

3.9.1 General

3.9.1.1 Applicability. Precast Concrete Full Depth Deck Panels (FDDP) as detailed in Part III of this Bridge Manual may be used for new bridge deck construction as well as for replacement of the existing bridge decks. The roadway profile grade shall not exceed 6%. Spray applied membrane waterproofing and a hot mix asphalt wearing surface shall be used on all bridge decks constructed with FDDP. Continuity connections of the Full Depth Deck Panels may be achieved either with longitudinal post tensioning or by using reinforced cast–in-place closure pours.

3.9.1.2 Structure Types. FDDP can be used on practically any bridge structure that is presently designed with the cast-in-place deck except NEXT F beams. Steel Stringers, Steel Girder/Floorbeam Systems, Steel Truss Systems, Precast Prestressed Concrete Beams (Box and NEBT), as well as Suspension and Cable Stayed Systems are the types of bridge structure that can be designed with FDDP.

3.9.1.3 Framing Geometry and Layout. For straight bridges FDDP shall be laid out as detailed in Part III of this Bridge Manual. They shall be set to match the cross slope of the finished roadway. For bridges with the superelevated deck, no roadway crown, and the total out-to-out width of the deck not exceeding 40 feet, a single precast deck panel can be used to cover the entire width of the bridge. To accommodate roadway crowns, stage construction joints, bridges with an out-to-out width greater than 40 feet, and bridge widening projects, longitudinal cast-in-place closure pour(s) should be used, to be detailed and constructed as per Part III of this Bridge Manual.

For horizontally curved bridges FDDP can be cast in a trapezoidal shape so that the transverse joints between the individual panels are radial to the curve.

3.9.2 Materials and Fabrication

3.9.2.1 Concrete Stresses. Standard size FDDP shall be made of 5000 HP cement concrete. If beam spacing exceeds 10 feet, pre-tensioning of the deck panels or an increase in their thickness may be required in order to amplify flexural resistance of the panels. If pre-tensioning is used, the design of the deck panels shall be based on a concrete compressive strength (f'_c) of 6,500 psi. The concrete compressive strength at release (f'_{ci}) shall be taken as 4,500 psi.

3.9.2.2 Prestressing Strands. Only Low Relaxation strands meeting the requirements of AASHTO M203 shall be used as required by MassDOT Specifications. Strands shall be 0.6” in diameter and shall not be epoxy coated. Typical strand pattern shall be laid out with zero eccentricity in order to resist the positive as well as negative moments in the panels and to minimize the cambering of the deck slabs after casting. FDDP shall be fabricated with the prestressing strand layout as shown on the Construction Drawings.

3.9.2.3 Mild Reinforcing Steel. All mild steel reinforcement shall be epoxy coated Grade 60 reinforcing steel.

3.9.2.4 Post-Tensioning Ducts and Anchorage Devices. Only 2” nominal diameter post-tensioning ducts with a maximum of four (4) 0.6”- diameter prestressing strands shall be used in construction of FDDP. Plastic and galvanized metal ducts are both acceptable. They shall be used in conjunction with the flat anchorage assemblies, which are to be located and detailed as per Part III of this Bridge Manual. When locating the anchorage assemblies, smaller horizontal dimensions measured from panel edges as well as from the edge of the shear connector block-outs can be used, provided that the anchorage forces are accounted for in the design of the panels.

3.9.2.5 Tolerances. FDDP fabrication and erection tolerances must be provided on the Construction Drawings. It is very important to have many of the fabrication tolerances measured from a common working point or line that is shown on the shop drawings. Center-to-center measurements can lead to a build-up of measuring errors and unacceptable results. Special attention should be given to the location of the longitudinal post-tensioning ducts. Misalignment of these ducts can cause considerable problems in the field. In addition, to avoid flexing of the plastic ducts during concrete placement, it is recommended to properly secure the ducts to the deck panel reinforcing or to stiffen them by attaching them to a parallel reinforcing bar.

3.9.3 General Design Requirements

3.9.3.1 General. The design of precast deck panels shall be the same as for a cast-in-place concrete deck. The “strip method” of design shall be used as per the *AASHTO LRFD.* Deck panels may be reinforced using mild reinforcement, prestressing, or a combination of both. The design of the deck shall follow normal sectional design requirements as per the *AASHTO LRFD.* The spacing of reinforcement shall account for the presence of shear connector block-outs, anchorage assemblies, and hand holes. Special care should be used for the detailing of deck overhangs. Barrier and overhang reinforcement details may need to be adjusted to account for the locations and sizes of shear connector block-outs.

3.9.3.2 Mild Reinforcement. All transverse (primary) and longitudinal mild reinforcement for FDDP shall be as per details provided in Part III of this Bridge Manual. The spacing of transverse and longitudinal steel will need to be adjusted to avoid interference with shear connector block-outs, leveling devices, etc. and to provide the proper cover.

3.9.3.3 Longitudinal Post-Tensioning. Longitudinal post-tensioning shall be used to provide compression between individual deck panels. The post-tensioning shall provide a minimum of 250 psi of prestress after short-term losses due to anchorage set friction, and elastic shortening. Time dependent losses in longitudinal post-tensioning need not be accounted for in the design. For continuous spans, the Designer shall design for and provide additional post-tensioning to overcome the tensile stress due to unfactored negative composite dead and live load moments and maintain a net prestress of 250 psi compression in the negative moment region. This may not be practical for long span bridges and alternate deck systems may need to be investigated. Since net compression is maintained in the deck, the Designer need not provide the one percent of the deck area steel requirement per the *AASHTO LRFD*.

The calculation of the post-tensioning forces including the number of strands shall be based on assumed values for friction, wobble and anchorage set as per the *AASHTO LRFD.* The minimum final post-tensioning force per tendon and the minimum effective prestress shall be shown on the plans, as well as a sequence for stressing the tendons (generally starting at the center and working to the outside). The plans shall note the assumptions used to develop the post-tensioning force including the assumptions used for loss calculations. The project specifications shall include requirements for submission of calculations for the design of the post-tensioning system. The final design of the post-tensioning is the responsibility of the Contractor and shall account for the hardware chosen for the construction of the deck.

The post-tensioning ducts on horizontally curved bridges with curved beams shall follow the roadway curvature. The ducts on bridges with minor curves and straight beams can be placed parallel to the girders. The design of the longitudinal post-tensioning for bridges with curved ducts shall take into account the friction losses in the post-tensioning ducts due to the curvature. In the case of large radius horizontal curves, it is acceptable to run the post-tensioning ducts straight within each individual deck panel combined with small angle points at the hand hole duct splice location.

3.9.3.4 Anchorage Zones. The design of the local zone reinforcement shall be the responsibility of the Contractor. The design of general zone reinforcement shall be the responsibility of the Designer. Local zone and general zone reinforcement shall be designed according to the *AASHTO LRFD.*

3.9.3.5 Composite Action. FDDP shall be made composite with the supporting beams. Composite action shall be achieved with shear connectors placed in block-outs in the deck panels. The design of the shear connectors shall be the same as for a cast-in-place concrete deck, except that the spacing of the shear connectors shall coincide with the spacing of the shear connector block-outs. Shear connectors shall consist of welded studs (for Steel Stringers, Prestressed Concrete Box and NEBT beams) or epoxy coated reinforcement extending from the top of the girders (for NEBT beams only) and shall be detailed as per Part III of this Bridge Manual.

3.9.3.6 Continuous Spans. The longitudinal post-tensioning may be used for the design of continuous girders in negative bending regions. The post-tensioning tendons may be accounted for in the calculation of the ultimate strength of the girder.

3.9.3.7 Girder Haunches. The design and detailing of the forming for the girder haunches is the responsibility of the Contractor and shall not be shown on the plans. The height of the girder haunches shall be the same as for cast-in-place concrete except on steel girder bridges with bolted splices. In this case, the height of the haunches may need to be increased to accommodate the splice plates and bolt heads. The nuts may be installed on the underside of the flange splice plate, provided that there is no conflict with the installation of the web splice bolts.

3.9.3.8 Handling. The design of lifting hardware and the handling stresses within the deck panel is the responsibility of the Contractor. Specifications shall require that lifting hardware shall be designed in accordance with the provisions of the latest edition of the PCI Design Handbook. The criteria for "no discernable cracking" shall be followed. The design for handling shall account for the presences of all block-outs.

3.9.4 Construction

3.9.4.1 Construction Sequence for post-tensioned FDDP. The sequence of construction for FDDP shall be such that the longitudinal post-tensioning is done after the transverse joints between the individual deck panels have been grouted and before they have been made composite with the girders. This sequence of construction assures that post-tensioning will not introduce additional bending moment into the girders, which could be detrimental to their performance. The sequence of construction shall be clearly outlined on the Construction Drawings.

3.9.4.2 FDDP Grade Elevations. The anticipated grade elevations of each corner of each deck panel after all deck panels are placed on a span and after all composite dead loads are applied must be provided on the Construction Drawings. These elevations are to be calculated as follows:

1. Calculate the theoretical finished roadway grade elevation directly over the deck panel at four (4) corners of each panel along the bridge span.
2. From these elevations subtract the total thickness of the wearing surface.
3. To the above elevations add the total dead load deflection due to all composite dead loads applied after the deck panels have been placed including, but not limited to the wearing surface, sidewalks, safety curbs, and rail/barrier systems. The result is the grade elevations of each deck panel corners that shall be provided on the Construction Drawings.

3.9.4.3 Vertical Adjustment. Vertical adjustment assemblies shall be used to assist in the equal deck panel weight distribution as well as to alter the grade elevations of the deck panels after their placement. The leveling devices shall be designed by the Contractor and shall meet the following criteria:

- The leveling devices shall be detailed so that all hardware that is to remain in place is set within a grouted recess with adequate cover.
- Portions of the leveling devices projecting from the deck, or not having adequate cover shall be removed after placement of the non-shrink grout.

3.9.4.4 Horizontal Adjustment. Horizontal adjustment of FDDP is achieved by providing the transverse joints between individual deck panels with a nominal width of ½". The width of these joints shall be adjusted in the field by ± 3/8" to account for fabrication and erection tolerances.

The layout of panels in the field shall be based off common working lines (transverse and longitudinal). The transverse alignment of panels shall be based on a longitudinal working line, not the deck edge or girder alignment. The use of girder lines for alignment is not recommended due to the fact that the girders may not be perfectly straight (but within tolerance) after installation.

3.10 DESIGN AND ANALYSIS OF INTEGRAL ABUTMENT BRIDGES

3.10.1 General

Integral abutment bridges (IAB) are single-span or multiple-span continuous structures with each abutment rigidly connected to the superstructure and supported by a single row of flexible vertical piles. The primary purpose of rigid connection is to eliminate the need for deck movement joints and bearings at abutments.

Integral abutment bridges differ from traditional rigid frame bridges in the manner which movement is accommodated. Rigid frame bridges resist the effects of temperature change, creep and shrinkage with full height abutment walls that are fixed or pinned at the footing level. The effects produced by longitudinal forces in integral abutment bridges are accommodated by designing the abutments to move with less induced strain, thus permitting the use of smaller and lighter abutments.

3.10.2 Loads, Load Factors, and Load Combinations

3.10.2.1 Permanent Loads. Permanent Loads on the abutments include the dead weight of the girders, deck and approach slab, integral wingwalls, intermediate diaphragms, and the abutment diaphragm. The weight of the wearing surface, sidewalks and safety curbs, barriers/railings, utilities, sign structures, lighting systems shall be included as well. All dead loads on the abutments shall be distributed equally to all piles.

3.10.2.2 Live Loads. The total Live Load on the abutment shall be determined assuming the largest number of traffic lanes that may be allowed by the total roadway width plus sidewalks. For the design of the abutments and the piles, live loads shall be equally distributed to all girders in the cross section. Multiple presence factors shall be applied. The dynamic load allowance shall be used for the pile cap design, but not for design of the piles.

For bridges with sidewalks, the following two cases are to be investigated and the most conservative shall be used.

1. Pedestrian load is ignored. The number of traffic lanes is calculated based on the total roadway width on the bridge, including the width of sidewalk(s) as if it was/were a part of the travelled way.
2. The number of traffic lanes is calculated based on the actual curb-to-curb width. Pedestrian load is applied to the abutment.

Centrifugal force shall be considered in the design of integral abutments of a curved bridge. It shall be calculated and applied as specified in Article 3.6.3 of the *AASHTO LRFD.*

Braking forces shall not be considered in the design of integral abutments because they are resisted by the soil forces acting on the rear face of the abutments.

3.10.2.3 Wind Loads. Transverse wind acting on the superstructure and on live load shall be considered in the design. The direction of the transverse wind force shall be taken perpendicular to the longitudinal axis of the bridge.

Wind load on the structure shall be calculated using the total superstructure thickness including the bridge barrier and shall be distributed to all piles equally.

Wind on live load shall be assumed to act at a distance of 6 feet above the bridge deck. Statics shall be used to determine the effect of this load on the piles by applying a moment about the longitudinal axis of the bridge at the base of the integral abutment cap. This approach shall be used to determine the increase and decrease in loading to the piles.

3.10.2.4 Thermal Movements. The thermal movements, δ_T, shall be calculated in accordance with Subsection 3.1.8 above. For simple spans with constant width and with both superstructure and substructure symmetric in the bridge elevation, the thermal movements at each integral abutment shall be taken as half the change in bridge total length due to uniform temperature change.

3.10.2.5 Secondary Loads. The creep and shrinkage movement should be addressed mostly in designs of cast-in-place or prestressed concrete superstructures. Except for the effect of creep and shrinkage on the vertical reactions of simple prestressed spans made continuous for live loads, abutment loads caused by creep, shrinkage, thermal gradient and differential settlements need only be considered for bridges longer than those specified in Subsection 3.10.11, for the Simplified Design Method.

3.10.2.6 Load Factors and Load Combinations. Load Factors and Load Combinations for integral abutment bridges shall be as per Article 3.4 of the *AASHTO LRFD*. The following should also apply:

- Passive earth pressure shall be as per Subsection 3.10.8;
- Thermal movement is a major source of loads on the abutment and abutment piles. Both the passive earth pressure on the abutment and the stresses in steel piles due to thermal movements are not reduced by the plastic flow of the concrete expected due to the seasonal nature of the thermal movements. Therefore, no reduction in the load factor for uniform temperature is allowed and a load factor of 1.0 is used all the time.
- Seismic design requirements shall be as specified for abutments in Section 3.4

3.10.3 Superstructure Types

Only Steel I-beams (rolled beams and plate girders), Prestressed Concrete Spread Deck and Box Beams, NEBT beams and NEDBT, NEXT F and NEXT D beams and CIP concrete slab bridges shall be used with integral abutment bridges.

3.10.4 Approach Slabs

Approach slabs shall be used for all integral abutment bridges. The approach slab shall be detailed to remain stationary by constructing a key away from the abutment and shall be detailed to allow sliding at the end supported by the abutment.

3.10.5 Abutment Backfill and Drainage

The area behind the abutments shall be backfilled with MassDOT's Gravel Borrow for Bridge Foundations and a drainage system shall be provided as shown in Chapter 15 of Part II of this Manual.

3.10.6 Construction of Integral Abutments

Integral abutments shall be constructed as shown in Chapter 15 of Part II and Chapter 3 of Part III of this Bridge Manual.

Construction Loads need to be considered in the pile cap design if construction equipment is allowed on the bridge before pouring the abutment diaphragm. In such cases, the load factors for Construction Loads shall be taken as per Article 3.4.2 of the *AASHTO LRFD*.

3.10.7 Superstructure Design Methodology

The connection between the beams and the abutment shall be assumed to be simply supported for superstructure design and analysis. It is recognized that, in some cases, it may be desirable to take advantage of the frame action in the superstructure design by assuming some degree of fixity. This, however, requires careful engineering judgment. Due to the uncertainty in the degree of fixity, frame action shall not be used to reduce design moments in the beams.

3.10.8 Pile Cap and Abutment Diaphragm Design

The superstructure is assumed to transfer moment, and vertical and horizontal forces due to all applicable loads, at the time when the rigid connection with the abutment is achieved. The effects of skew, curvature, thermal expansion of the superstructure, and roadway grade are considered.

The design provisions below are conservative because the pile cap and the abutment diaphragm are very rigid members, therefore all loads shall be uniformly distributed across the abutment.

The abutment shall be designed for the following two cases:

1. The pile cap is designed to resist all vertical loads including live load. It is assumed to act as a continuous beam supported by piles. The analysis can be simplified by assuming the pile cap acting as a simple span between piles and then taking 80% of simple span moments to account for continuity. Shears may be taken equal to simple span shears. Due to the relatively large dimensions of the abutment walls, minimum reinforcement is usually sufficient to satisfy the strength requirements.
2. The entire abutment wall (the combined height of the pile cap and the abutment diaphragm) is designed to resist the earth pressure due to the backfill material, assuming the wall to act as a horizontal continuous beam supported on the girders, i.e., with spans equal to the girder spacing along the skew (if any).

The abutments should be kept as short as possible to reduce the magnitude of soil pressure developed. A minimum of 3'-0" for inspection access shall be provided. A minimum fill cover over the bottom of the abutment of 3'-0" is desirable. It is recommended to have abutments of equal height due to the fact that a difference in abutment heights causes more movements to take place at the shorter abutment. Abutments of unequal height shall be designed by balancing the earth pressure consistent with the magnitude of the displacement at each abutment.

The magnitude of lateral earth pressure developed by the backfill is dependent on the relative wall displacement, δ_T/H, and may be considered to develop between full passive and at-rest earth pressure. The backfill force shall be determined based on the movement-dependent coefficient of earth pressure (K). Results from full scale wall tests performed by UMASS[1] show reasonable agreement between the predicted average passive earth pressure response of MassDOT's standard compacted gravel borrow and the curves of K versus δ_T/H for dense sand found in design manuals DM-7[2] and NCHRP[3]. For the design of integral abutments, the coefficient of horizontal earth pressure when using compacted gravel borrow backfill shall be estimated using the equation:

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$

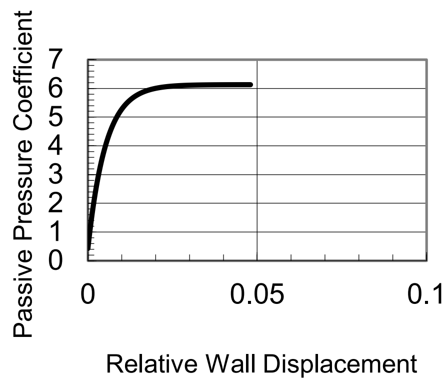


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ T/H

The longitudinal reinforcement of the pile cap has been predesigned and is provided in the Reinforcement Selection Tables of Chapter 15 of Part II of this Bridge Manual. It represents an upper bound for the required reinforcement assuming the girders are located at the positions that produce maximum effects on the pile cap and assuming a conservative value of other dead loads on the abutment wall.

Stirrups intended to resist horizontal shear forces acting on the pile cap due to soil passive pressure shall be provided as shown in Chapter 15 of Part II of this Bridge Manual.

L-shaped connection reinforcing bars indicated in the standard drawings of Chapter 15 of Part II of this Bridge Manual are provided to transfer the maximum expected connection moment between the abutment and the superstructure. These bars shall be #6 @ 9" for girders up to 8 feet deep. For deeper girders they shall be designed. The vertical leg of the connection bars shall be placed as close as practical to the back face of the abutment. The horizontal leg shall be extended into the deck beyond the inside face of the abutment diaphragm at the elevation of the deck top longitudinal reinforcement for a length equal to 10% of the span plus the development length, for simple span bridges. For continuous span bridges the bars shall be extended to 10% of the end span plus the development length.

Refer to Chapter 15 of Part II of this Manual for details of the integral abutment reinforcement.

3.10.9 Integral Wingwall Design

Only U - shaped (parallel to the longitudinal axis of the bridge) integral wingwalls shall be used between the abutments and the Highway Guardrail Transitions. The length of the integral wingwalls shall be as required by site and bridge geometry, with a minimum and maximum length of 2 feet and 10 feet, respectively. When a longer wingwall is required a combination of integral and independent wingwalls shall be used.

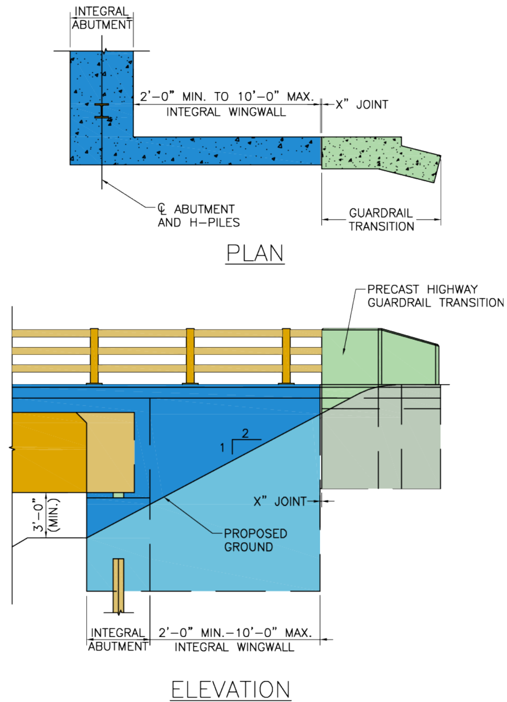


Figure 3.10.9-1: Integral Wingwall Geometry

A parametric study [19] was performed to determine the required primary integral wingwall reinforcement (parallel to the longitudinal axis of the bridge). It was designed based on the moment taken as a sum of the factored active earth pressure moment and the moment resulted from the vehicular collision force applied at the top of the barrier/railing. This load case is considered under Extreme Event II Load Combination with the load factors of 1.5 and 1.0 for the active earth pressure and the vehicular collision, respectively. The analyses were performed for all types of the barrier/railing used by MassDOT as per Chapter 12 of Part II of this Bridge Manual.

Based on the above parametric study, the minimum required primary integral wingwall reinforcement (longitudinal) was determined and shall be as per design table of Chapter 15 of Part II of this Bridge Manual.

The secondary integral wingwall reinforcement (vertical) was determined based upon the shrinkage requirements of the *AASHTO LRFD* and is provided in the design table of Chapter 15 of Part II of this Bridge Manual, as well.

3.10.10 Piles

3.10.10.1 General. The abutment shall be supported on a single row of vertical H-piles with the webs oriented parallel to the centerline of the abutment regardless of the skew. The permissible total length of integral abutment bridges is sensitive to the relative slenderness of the pile section. There are only two H-pile sections that satisfy the provisions of *AASHTO LRFD* Article and are capable of developing a fully plastic stress distribution and may be used where plastic hinge formation is expected. These sections are HP10X57 and HP12X84 and they shall be used exclusively in integral abutment construction. Only Grade 50 steel (Fy = 50 ksi) shall be used for the above H-pile sections.

3.10.10.2 The pile tip elevation shall be established as per requirements of Subsection 3.10.11 below. When scour is anticipated, the minimum pile length, to meet the structural and geotechnical resistance as required by Subsection 3.2.10, shall be provided beyond the depth of computed total scour.

3.10.10.3 The minimum and maximum distances between the pile flange and the end of the abutment, measured along the skew, shall be 18” and 3'-0", respectively (these limits shall not apply to the staged construction). The piles shall be embedded 2 feet into the pile cap. Maximum pile spacing for integral abutment piles shall be 10 feet. The minimum pile spacing should not be less than 3’-6”. A minimum of one (1) pile per beam line at each abutment shall be used.

3.10.10.4 A trench with a depth of 3’ and a minimum width of 2’-6” shall be constructed directly below the bottom of the pile cap. After the piles are driven, the trench shall be filled with crushed stone to reduce the resistance to the lateral pile movement due to the thermal forces resulting from the temperature changes.

3.10.11 Pile Design

3.10.11.1 General. MassDOT has two methods for Designers to use in designing Integral Abutment piles. Both employ the same pile design methodology, however in the Simplified Method, the thermal movement and skew effects have been factored into the Maximum Factored Axial Compressive Resistance per pile that is provided to Designers to use, while for the 3D Space Frame Analysis Method, the Designer must perform the pile design based on the load effects and displacements obtained from a 3D model of the bridge structure.

1. **The Simplified Method**. This method allows designers to determine an adequate H-pile section (HP10X57 or HP12X84) by calculating only factored gravity loads (dead and live) for the anticipated number of the abutment piles for a given bridge. The Simplified Method as described below shall be used only if ***all*** of the following boundary conditions are satisfied:
   - Total bridge lengths shall be limited to 140 feet for steel bridges and 200 feet for concrete bridges. These maximum span lengths restrict the lateral pile’s head displacement to approximately ½” of the one-way movement.
   - Skew angles shall be limited to 30˚.
   - The structure shall be a straight bridge or a curved bridge with straight beams that parallel with each other.
   - Horizontal curvature shall be limited to a 5˚ subtended central angle.
   - The difference in the profile grade elevation at each of the abutments shall not exceed 5% of the bridge length.
   - Abutment heights, measured from the deck surface to the bottom of the cap, shall not exceed 15 feet.
   - The bridge shall set upon parallel abutments and piers.
   - The bridge shall have abutments with parallel wingwalls (U-wingwalls).

- The top of bedrock, as per Geotechnical Report, shall be located lower than the established pile tip elevation.
- The abutments of the bridge are not scour susceptible.

2. **The 3D Space Frame Analysis Method**. This method shall be used when one or more of the specified above boundary conditions are not met, as well as for integral bridges with unique or unusual geometry. It requires the modeling of the entire bridge structure using an "equivalent length" of unsupported fixed end pile based on the top of the pile deflection required for thermal movement, as described below.

3.10.11.2 Pile Tip Elevation. In order to obtain the intended pile behavior, the piles must be installed to the point of fixity (Lf) or deeper. Therefore, the final pile tip elevation to be shown on the Construction Drawings shall be the longer of the following calculated pile lengths:

1. The required pile length based on the Factored Geotechnical Pile Resistance.
2. The required pile length, based on the theoretical depth to pile fixity, L_f, plus five (5) feet. The theoretical depth to pile fixity is defined as the depth along the pile to the second point of zero lateral deflection, in relation with the calculated thermal movement of the bridge superstructure as shown in Figure 3.10.11-1 below. Furthermore, the pile must be installed to a depth of five (5) feet below the point of the theoretical pile fixity, to account for any uncertainly in the actual L_f.

At locations where bedrock is situated below the estimated pile tip elevation, but is in close proximity, the piles may be extended to the top of rock. In such cases the number and size of the piles per abutment need only be based on the Design Factored Structural Resistance of the piles. The requirements of the Subsection 3.10.10 shall also be followed.

At locations where the bedrock elevation is less than five (5) feet below the elevation of the theoretical point of pile fixity, the piles need only be driven to bedrock. At locations where the bedrock is located at an elevation that is higher than elevation of the theoretical point of pile fixity, the site is considered unsuitable for pile supported integral abutments. At locations where the bedrock profile is uncertain, i.e. the borings produce significant discrepancy in the top of rock elevations, geophysical subsurface testing methods may be used to establish the profile with the improved reliability to determine if the site is suitable for pile supported integral abutments or not.

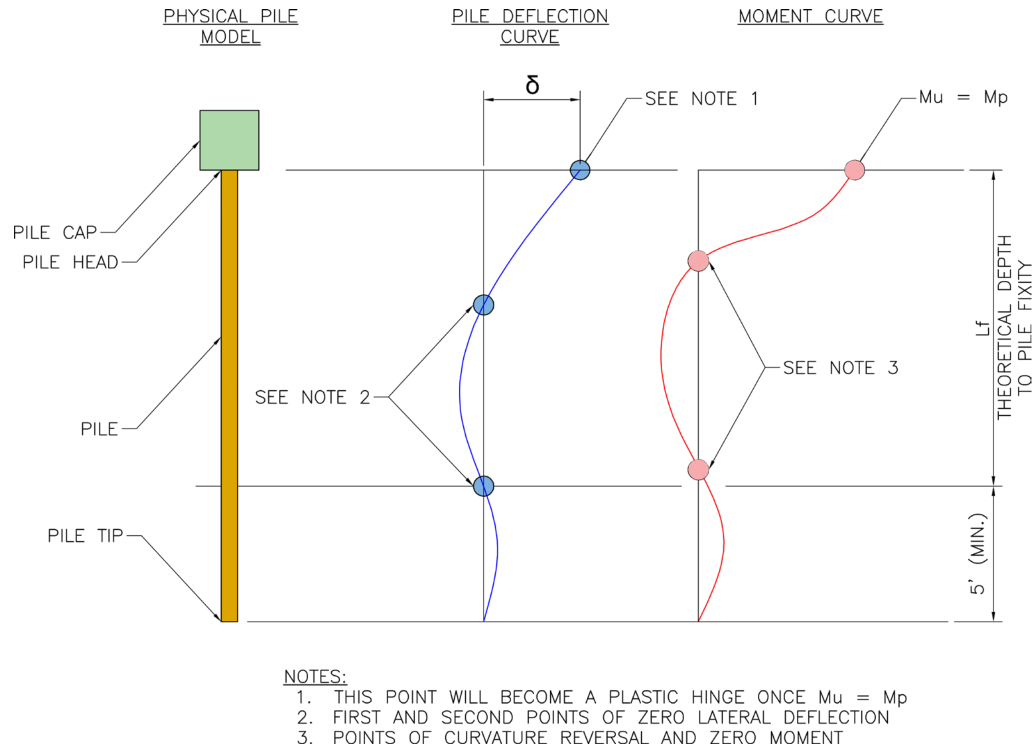


Figure 3.10.11-1: Integral Abutment's Pile Design Model when $M_u = M_p$

3.10.11.3 General Pile Design Methodology. Both design methods are based on the methodology which incorporates the provisions contained in the *AASHTO LRFD* and the following:

Integral abutment piles are considered to be fully braced against lateral torsional buckling and gross Euler buckling, except when checking scour.

The Factored Geotechnical Pile Resistance and the Factored Axial Pile Resistance shall be computed and evaluated based on the following three (3) controlling cases:

1. Geotechnical Factored Resistance of the pile to transfer load to the ground.
2. Geotechnical Factored Resistance of the ground to support the load.
3. Factored Axial Resistance of the pile according to the procedures outlined in the *AASHTO LRFD.*

Six different soil types, which were considered for evaluation, as well as their properties, are specified in the following table:

Table 3.10.11-1: Soil Properties

Soil Type		Unit Weight γ {pcf}	Friction Angle ϕ {degrees}	Modulus k {pci}	Cohesion c {psf}	Strain ε_{50}
1	Dry Loose Sand	122	30	25	-	-
2	Wet Loose Sand	60	30	20	-	-
3	Dry Dense Sand	138	40	225	-	-
4	Wet Dense Sand	76	40	125	-	-
5	Wet Stiff Clay	70	-	500	2600	0.005
6	Wet Soft Clay	70	-	100	575	0.02

In developing the pile capacities and depth to theoretical point of fixity for the Simplified Method, the L-Pile computer program was used to induce a displacement of ½" along the longitudinal axis of the bridge at the pile head for both HP10X57 and HP12X84 pile sections. The resulting moments from the lateral translation at the pile head were established. Subsequently, the Maximum Factored Axial Resistance of a pile sections was calculated per Article 6.9.2.2 of the *AASHTO LRFD* by solving the interaction Equation 6.9.2.2-2 for the Axial Compressive Load, P_u, with the following modification: due to the fact that compact sections are capable of developing a fully plastic stress distribution and have an inelastic rotational capacity of 3 before the onset of flange local buckling, the final design procedure for compact pile sections incorporates an inelastic rotational capacity factor θ_i = 1.75 to account for the pile's ability to undergo inelastic rotation (for weak axis bending only) and the associated increase in pile head translation. The modified interaction equation used in the pile analysis is:

$$\frac{P_u}{P_r} + \frac{8}{9}\left[\frac{M_{uy}}{\theta_i M_{ry}} + \frac{M_{ux}}{M_{rx}}\right] \leq 1.0$$

Where:

P_u = Factored axial compressive load;

P_r = Factored axial compressive resistance of the respective pile section;

$P_r = \phi_c P_n = \phi_c F_y A_g$ = axial compressive resistance of a compact pile section that is fully braced against sidesway and buckling, where:

ϕ_c = 0.70 = Resistance factor for compression per *AASHTO LRFD* for H-Piles with combined axial and flexural resistance.

M_{uy} ; M_{ux} = Factored flexural moment about the pile's respective axis determined from analysis;

M_{ry} ; M_{rx} = Factored flexural resistance about the pile's respective axis;

For compact sections these values shall be taken as:

$M_{ry} = Z_y F_y$ and $M_{rx} = Z_x F_y$

Where:

Z_y ; Z_x - Plastic section modulus for respective axis;

$Z \leq 1.5$ S (AISC[7]);

S = Section modulus for the pile's respective axis;

θ_I = Coefficient of inelastic rotational capacity (see above).

In accordance with Paragraph 3.2.10.2 of this Bridge Manual, for the design of integral abutment piles at the scour design flood for scour depth, or in the case where the piles penetrate through extremely soft material such as peat, the pile unbraced length must be considered as part of the design. In these cases, M_{rx}, M_{ry} and P_r shall be used per *AASHTO LRFD*, while considering an unbraced length > 0 and using interaction equation contained herein. Pile unbraced length may be taken as the exposed depth of pile or the depth of extremely soft material.

In accordance with Paragraph 3.2.10.5 of this Bridge Manual, for the design of integral abutment piles at the scour check flood for scour depth, piles shall be checked for axial load only, with consideration of unbraced length, following all requirements in *AASHTO LRFD*.

In skewed bridges the deflection due to thermal movement was resolved into components and the piles were analyzed for bi-axial bending. Skew effects are included in the values provided in the Design Tables below and the Designer may interpolate between them for a given skew, if needed. It should be noted that since the load factor for thermal loads is 1.0, the results from the L-Pile analysis were used as factored loads.

The P-Δ effects of the axial load were investigated using the L-Pile program and proven to have an insignificant effect on the total bending in the pile section for the deflections due to one-way thermal movements for up to ½". As a result, the P-Δ effects were ignored in the development of the Simplified Method. For the 3D Space Frame Analysis Method with one-way thermal movements larger than ½" the P-Δ moments (Axial Load x Deflection) shall be included.

In an effort to more accurately simulate field conditions, an analytical study was performed for each type of soil using the L-Pile program with 5 feet of overburden located above the top of the pile head. This minimum depth of overburden was included because in all instances the top of the pile is at least 5 feet below the surface of the roadway. The layer below the overburden was a 3-foot thick layer of crushed stone, followed by the soil types described in Table 3.10.11-1. The results of the analysis showed that the effects of the overburden did not have a significant effect on the behavior of the pile since the 3' crushed stone filled trench allows the pile head to translate relatively freely for these anticipated thermal movements. Therefore, overburden need not be considered and the tabulated values in Paragraph 3.10.11.4 do not include overburden.

The L-Pile results were used to determine the theoretical depth to pile fixity for each soil type specified above for the enforced lateral displacement as well as determine the bending moment at the top of the pile for fixed head conditions.

3.10.11.4 The Simplified Design Method Procedure. Assuming anticipated number of piles (one pile per beam line) at each abutment, the Design Factored Axial Compressive Load per pile shall be computed based on gravity loads (dead and live) only. In order to establish an adequate pile section, the computed load shall be checked against the Maximum Factored Axial Compressive Load per pile, Pu, shown in the Design Tables 3.10.11-2 and 3.10.11-3 below. The Design Tables provide the Designer with the theoretical depth to pile fixity as well, which shall be used to establish final pile tip elevation as described in Paragraph 3.10.11.2 above.

Table 3.10.11-2: Maximum Factored Axial Load per pile and Theoretical Depth to Pile Fixity for HP 10X57 Pile Section

Soil Type		Skew				L_f (feet)
		0°	10°	20°	30°	
		P_u (kips)	P_u (kips)	P_u (kips)	P_u (kips)	
1	Dry Loose Sand	420	378	342	312	26
2	Wet Loose Sand	429	390	356	330	28
3	Dry Dense Sand	367	295	242	203	19
4	Wet Dense Sand	378	312	263	226	20
5	Wet Stiff Clay	380	296	248	214	16
6	Wet Soft Clay	471	428	403	385	26

Table 3.10.11-3: Maximum Factored Axial Load per pile and Theoretical Depth to Pile Fixity for HP 12X84 Pile Section

Soil Type		Skew				L_f (feet)
		0°	10°	20°	30°	
		P_u (kips)	P_u (kips)	P_u (kips)	P_u (kips)	
1	Dry Loose Sand	641	586	538	500	31
2	Wet Loose Sand	654	603	560	525	32
3	Dry Dense Sand	561	461	389	336	22
4	Wet Dense Sand	580	488	423	374	23
5	Wet Stiff Clay	583	468	405	361	18
6	Wet Soft Clay	713	657	625	603	30

3.10.11.5 The Finite Element Design Method Procedure. The initial choice of pile section shall be based on the Design Factored Axial loads as per procedure specified above for the Simplified Design Method.

The purpose of modeling the structure, as outlined below is to determine:

- Moments in the piles and in the abutment due to thermal and skew and scour effects
- Distribution of seismic loads on multi-span structures

A bridge shall be modeled as 3D space frame that includes, as a minimum, a "stick" model of the superstructure, abutments, wingwalls, piers (if any), piles, soils springs, and shall be representative of the geometry, including skew (refer to Figure 3.10.11-2 below). The frame elements representing the superstructure and any piers shall be modeled with transformed section properties located at the respective centers of gravity. The frame elements representing the abutments and rigid connectors shall be modeled with "infinite stiffness". The piles shall be modeled with their respective properties and rotated to align with the abutment skew. Soil springs shall be modeled and located as discussed below.

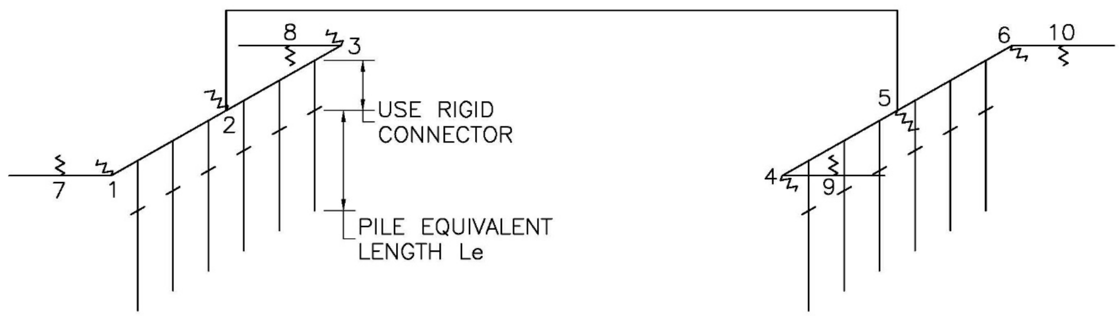


Figure 3.10.11-2: "Stick" Model Geometry

The soil behind the abutments shall be modeled with at least three (3) horizontal non-linear springs that are oriented perpendicular to the wall face with each of the springs located at 1/3 the height of the abutment wall from the base, see nodes 1 through 6 in the diagram above. In addition, the nodes 2 and 5 should be located at mid length of the walls and nodes 1, 3, 4, and 6 should be located at the ends of the walls. The soil spring stiffness behind each abutment shall be distributed based on the tributary area for the middle portion equal to 50% at nodes 2 and 5 and end quarters equal to 25% at nodes 1, 3, 4, and 6. The non-linear soil spring stiffness shall be based on K values determined in accordance with Subsection 3.10.8 above for assumed incremental displacements. The soil springs shall not carry tension forces. The same K values shall be used for both static and dynamic loads. Similarly, the soil behind the integral wingwalls shall be modeled as a horizontal soil spring located at the half point from the wingwall end and at 1/3 the height from the wall base, nodes 7, 8, 9 and 10 in the diagram above, with stiffness calculated as stated above. The choice of the vertical location of the pressure resultant for placing the soil springs is based on a classic triangular soil pressure distribution. To capture the full height of the abutment wall as it relates to the piles, rigid connectors should be used to connect the pile tops at the base of the wall to the horizontal frame member representing the abutment which is located at the abutment 1/3 point.

For additional information on modeling non-linear behavior refer to Article 4.5.3.2.1 of the *AASHTO LRFD*.

The length of pile from the base of the abutment to the point of fixity shall be the equivalent length, L_e, defined as the theoretical equivalent length of a free standing column with fixed/fixed support conditions translated through a pile head horizontal displacement δ_T. The equivalent length for each pile, L_e, used in the 3D model shall be as outlined in Table 3.10.11-4 below.

Table 3.10.11-4: Equivalent Pile Length (L_e)

Soil Type		L_e (ft)	
		HP10x57	HP12x84
1	Dry Loose Sand	8.3	9.5
2	Wet Loose Sand	8.5	9.8
3	Dry Dense Sand	7.3	8.3
4	Wet Dense Sand	7.5	8.5
5	Wet Stiff Clay	7.5	8.5
6	Wet Soft Clay	10	11.5

In order to obtain the pile behavior associated with the calculated equivalent lengths, the piles must be installed to a point of fixity or deeper. As defined in Paragraph 3.10.11.2, the theoretical depth to pile fixity is defined as the depth along the pile to the second point of zero lateral deflection. The required length of fixity, L_f, shown in Tables 3.10.11-2 and 3.10.11-3 was converted to the equivalent lengths, L_e summarized in Table 3.10.11-4 above. Equivalent length, L_e, is the length of a free- standing column with fixed/fixed support conditions translated through a pile head horizontal displacement δ_T. The pile deflection and moment diagrams associated with the above behavior are shown in Figure 3.10.11-1. The pile tip elevations shall be determined in accordance with Paragraph 3.10.11.2.

To calculate the Factored Axial Pile Resistance the analyses shall be performed for all applicable Load Combination Limit States per the *AASHTO LRFD*.

If the analysis results indicate that the piles are inadequate, the Designer shall increase the pile size and/or add additional piles and re-analyze until an adequate pile size and/or spacing is determined.

3.10.12 References

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10. Commonwealth of Massachusetts, Massachusetts Highway Department, Geotechnical Section Internal Report, "A Preliminary Report on the Geotechnical Aspects of Integral Abutment Bridges", 1998.
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15. American Iron and Steel Institute and the Tennessee Department of Transportation, "Highway Structures Design Handbook, Volume II, Chapter 5, Integral Abutments for Steel Bridges", 1996.
16. VTrans Structures Section, "Integral Abutment Bridge Design Guidelines", 2nd Edition, 2008.
17. Pennsylvania Department of Transportation, Design Manual, Part 4, Volume 2, Appendix G - Integral Abutments, June 2008.
18. Modjeski and Masters, Inc., "Guidance on the Design for Vehicular Collision with Wingwalls", 2009.

3.11 REHABILITATION OF STRUCTURES

3.11.1 General Requirements

Every bridge rehabilitation project shall ensure a bridge structure that meets current code and load capacity provisions. Where feasible, structures shall be made jointless.

3.11.2 Options for Increasing Carrying Capacity

3.11.2.1 General. The following are traditional options for increasing the resistance of existing primary members. They can be used independently or in combination to achieve the desired effect. Not every structure can be upgraded using these options and therefore, sound engineering judgment should be employed when evaluating them.

1. Where the existing beams are of non-composite construction, redesigning the beams for composite action and providing for the addition of shear connectors may be sufficient to increase the carrying capacity.
2. Using a full depth HPC deck with a ¾" thick integral wearing surface may be used in lieu of a regular deck with a bituminous concrete wearing surface to reduce the added dead load. Thin HPC overlays shall not be considered due to the potential for constructability problems.
3. Using lightweight concrete for the deck instead of regular weight concrete. When using lightweight concrete, the Designer must take into account the reduced Modulus of Elasticity in the calculation of composite section properties as well as the increase in the development and lap lengths for reinforcing bars, as specified in the *AASHTO LRFD*.
4. On rolled steel beam sections, adding cover plates. On bridges with existing cover plates, consideration can be given to adding additional cover plates on the top of the bottom flange. This is usually accomplished by adding two small plates to the top of the bottom flange, placed

symmetrically either side of the web plate. Addition of any cover plates to an existing structure changes the stress distribution in the beam which must be accounted for in design, e.g. the bottom flange carries dead load stresses while the added cover plate is unstressed.

5. Where existing members have cover plates on the bottom flange, it is usually not economically feasible to remove them, especially if the bridge is over a road that has a high ADT.
6. Construct continuity retrofit of simply supported main members over the pier(s) in order to reduce live load stresses in positive moment region(s).

3.11.2.2 The standard 1½" haunch shall not be used in calculating composite section properties. However, where an excessive haunch depth occurs due to changes in bridge cross slope or changes in vertical profile, the haunch depth in excess of the standard haunch can be utilized in calculating composite section properties. For example, if the profile change results in a 6" haunch, the excess 4½" may be used in calculating section properties.

3.11.3 Fatigue Retrofits

3.11.3.1 All fatigue-susceptible details shall be fully investigated in bridge rehabilitation projects. Of particular concern are the ends of cover plates where a fatigue category E or E′ exists. In most cases, older cover plated beams will not meet current fatigue requirements for allowable stress ranges.

3.11.3.2 Reference is made to the *AASHTO LRFD* and the *AASHTO Manual for Bridge Evaluation*, Section 7, Fatigue Evaluation of Steel Bridges, for evaluating the remaining fatigue life of existing steel members.

3.11.3.3 For existing rolled beams with partial length cover plates, if the remaining fatigue life is inadequate or if cracks are found at the cover plate ends during a visual inspection, the beams will be retrofitted by installing splice plates on the bottom flange which will span over the cover plate end. These splices will be designed for the maximum force in the cover plate based on the cross sectional area and the stress in the cover plate under the Service and Strength Limit States. The splices will be designed as bolted slip-critical connections.

Installing bolts through the existing cover plate termination is not acceptable as a retrofit because it does not span over the cover plate end and does not relieve the stress riser associated with the transverse weld. Furthermore, if a crack at the end of the cover plate that was invisible at the time of the inspection were to grow and propagate through the beam flange, the bolts would not keep the beam flange from separating in the way a splice would.

3.12 ANCILLARY STRUCTURES

3.12.1 Pedestrian Bridges

Bridges whose primary function is to carry pedestrians, bicyclists, equestrian riders, and light maintenance vehicles shall be designed in accordance with the *AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges*. Pedestrian bridges shall be designed to comply with the Americans with Disabilities Act (ADA) law.

3.12.2 Temporary Bridges

Pre-Engineered Temporary Panelized Bridges are to be used wherever feasible to maintain traffic flow during bridge reconstruction projects. The design of Pre-Engineered Temporary Panelized Bridge superstructures shall be performed by the supplier and shall be reviewed and approved by the Designer. Where the use of Pre-Engineered Temporary Panelized Bridge superstructures is not feasible, all elements of the temporary bridge structure shall be designed by the Designer. The design of all temporary bridge substructures that are to be used by the public during a bridge project shall be the responsibility of the Designer. Temporary bridge substructures that support Pre-Engineered Temporary Panelized Bridges shall be designed for assumed loads from the superstructure. The temporary bridge substructures shall be located and detailed on the bridge Construction Drawings. The assumed vertical and horizontal geometry of the Pre-Engineered Temporary Panelized Bridge and the assumed design loads for the substructure shall be specified on the bridge Construction Drawings. All temporary bridge structures shall be designed as if the structure was intended to be a permanent installation. Provisions for seismic design may be waived with the approval of the State Bridge Engineer.

3.12.3 Sign Attachments to Bridges and Walls

3.12.3.1 General. Signs should not be attached to bridges. If this is unavoidable, then the following provisions shall apply to the design of the sign supports and their attachment to the bridge.

3.12.3.2 All sign attachments, their connections and their appurtenances shall be designed in accordance with the latest version, including current interims, of the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*. The effect of loads from the sign structure on the bridge structure in conjunction with the bridge dead and live loads shall be considered during design.

3.12.3.3 In the design of sign supports, the wind velocity to be used shall be in accordance with the basic wind speed figure contained in the latest version, including current interims, of the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals*.

3.12.3.4 When considering whether to attach a sign to an existing bridge structure, the following recommendations shall be observed:

1. Avoid attaching large signs to existing bridges (signs whose height is greater than 1.5 times the depth of the bridge beam plus coping height.
2. Avoid attaching signs to bridges where the angle between the sign face and the bridge fascia would exceed 30°.
3. Do not attach changeable message signs to existing bridge structures under any circumstances. These shall always be mounted on independent full span structures.
4. Even if it still seems more efficient to mount a sign on an existing bridge, the bridge must still be checked to verify that the beams can carry all of the sign loads (dead load, eccentric torsional load, out of plane bending, etc.) without global or local overstress. If members are overstressed then a retrofit design must be provided. Also, the condition of both the beam and the coping concrete must be investigated to verify that it is competent to be attached to.
5. Signs shall not be attached to bridges with prestressed concrete beams that would require field drilling for the sign attachments. Field drilling into prestressed beams is prohibited since the prestressing strands are embedded in the beams and careless drilling can sever the strands and reduce the load carrying capacity of the beam. Concrete inserts, if used, shall be cast into the beam during fabrication.
6. Expansion bolts embedded into existing copings shall have a minimum diameter of ¾”.

3.12.3.5 Sign supports shall be fabricated from steel conforming to AASHTO M 270 Grade 50 and shall be galvanized. All steel hardware shall also be galvanized.

3.12.3.6 The minimum size of angles to be used shall be L3x3x5/16. The minimum size weld to be used shall be ¼”.

3.12.3.7 The distance between sign support panels shall be selected so that the maximum positive and maximum negative moments in the panels shall be approximately equal. The bottom of the sign panel shall be a minimum of 6” above the bottom of the stringer.

3.13 BRIDGE INSPECTION

3.13.1 Bridge Inspection Access

3.13.1.1 The Designer should have provided for bridge inspection access during the preliminary engineering phase of the project in accordance with the guidelines set forth in Subsection 2.3.3 of Part I of this Bridge Manual. However, if during the final design phase, issues arise that may adversely impact the previously established inspection access provisions, the Designer shall consult with the MassDOT Bridge Inspection Unit for guidance on how to best resolve these issues in order to ensure adequate and safe inspection access.

3.13.2 Fracture Critical Bridge Inspection Procedures

3.13.2.1 If a bridge is designed with fracture critical members, the Designer must prepare and submit a Fracture Critical Inspection Procedure as part of the design process in addition to the contract documents. This procedure will be used to properly inspect these structures in accordance with federal regulations, 23 CFR Part 650, Subpart C, §650.313 (f).

3.13.2.2 The Fracture Critical Inspection Procedure shall be prepared on standard MassDOT forms as supplied by the Bridge Inspection Unit and shall consist of the following parts:

1. Index
2. Identification of Fracture Critical Members
   Where Fracture Critical portions of members (such as tension zones of non-redundant plate girders or floorbeams) exist, the entirety of the member is considered as a Fracture Critical Member. Identify these members both by text and visually by using key Construction Drawings, diagrams and elevation views of members. This list will be used by the inspectors to identify and inspect all Fracture Critical members on the bridge. The required inspection frequency shall also be noted.
3. Identification of Fatigue Sensitive Details
   Identify all Fatigue Sensitive details, including constraint-induced fracture prone details, on the Fracture Critical members both by text and through the use of the standard Fatigue Sensitive category diagrams. This list will be used by inspectors to identify and inspect all Fatigue Sensitive details on the Fracture Critical members. The required inspection frequency shall also be noted.
4. Inspection Procedure for Inspection of Fracture Critical Members
   Outline the procedure the inspectors are to follow when inspecting Fracture Critical members. The required inspection frequency shall also be noted.

5. Inspection Procedure for Inspection of Fatigue Sensitive Details
   Outline the procedure the inspectors are to follow when inspecting Fatigue Sensitive details or other details known to be succeptible to fracture, such as constraint-induced fracture prone details. The required inspection frequency shall also be noted.
6. Photographs
   Provide inventory photographs of the bridge structure and photographs of the typical Fracture Critical members and Fatigue Sensitive details for identification purposes.

The Federal Highway Administration Report No. FHWA-IP-86-26, "Inspection of Fracture Critical Bridge Members", dated September 1986, can be used as a reference and guide in preparing the inspection procedures of parts 3 and 4.

3.13.2.3 Since a Fracture Critical Inspection requires a very detailed, close visual "hands-on" inspection as a means of detecting cracks, the Designer shall make sure that all Fracture Critical members of the bridge can be accessed in accordance with Subsection 3.13.1.

3.13.3 Item 113 Coding for Bridges over Water

3.13.3.1 Designers are required to provide the Department with the applicable coding for Item 113 - *Scour Critical Bridges*. The Item 113 code supplied shall be in conformance with latest interims/errata of the Federal Highway Administration's *Recording and Coding Guide for the Structure Inventory and Appraisal of Nation's Bridges*. An Item 113 Code is required for all proposed bridge replacements, superstructure replacement and bridge rehabilitation projects that are over water.

At the completion of the construction of the project and receipt of the Initial Inspection Report, the Designer shall submit the completed *Designer's Coding of Item 113 – Scour Critical Bridges* form as shown in Figure 3.13.3-1.

This form certifies that the structure was built in conformance with the details shown on the construction drawing and that any design changes made during construction have been incorporated into this final scour code determination. This form is to be sent to the State Bridge Engineer for updating the bridge's *Structures Inventory and Appraisal* (SI&A) sheet.

DESIGNER'S CODING OF
ITEM 113 – SCOUR CRITICAL BRIDGES

CITY/TOWN: ______
DISTRICT: ______
ROADWAY ON BRIDGE: ______
FEATURE INTERSECTED: ______
BRIDGE IDENTIFICATION NO. (AS SHOWN ON CONSTRUCTION DRAWINGS):
BRIDGE NO.: ______ **BIN:** ______

In conformance with Subsection 3.13.3 of the LRFD Bridge Manual, we recommend as the Designer of Record for the structure listed above, the following Item 113 code of ______.

Designer Signature *Date*

CONSULATNT FIRM NAME: ______

(Consultants are directed to complete this form after the bridge is constructed and send the signed document to the State Bridge Engineer)

*Table information below from "Errata Sheet for Coding Guide 06/2011"

CODE	Description
N	Bridge not over waterway.
U	Bridge with "unknown" foundation that has not been evaluated for scour. Until risk can be determined, a plan of action should be developed and implemented to reduce the risk to users from a bridge failure during and immediately after a flood event (see HEC 23).
T	Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections until an evaluation is performed ("Unknown" foundations in "tidal" waters should be coded U.)
9	Bridge foundations (including piles) on dry land well above flood water elevations.
8	Bridge foundations determined to be stable for the assessed or calculated scour condition. Scour is determined to be above top of footing (Example A) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge[4] by calculation or by installation of properly designed countermeasures (see HEC 23).
7	Countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a plan of action have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event.
6	Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for assessed or calculated scour condition. Scour is determined to be within the limits of footing or piles (Example B) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge), by calculations or by installation of properly designed countermeasures (see HEC 23).
4	Bridge foundations determined to be stable for assessed or calculated scour conditions; field review indicates action is required to protect exposed foundations (see HEC 23).
3	Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions: – Scour within limits of footing or piles. (Example B) – Scour below spread-footing base or pile tips. (Example C)
2	Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations, which are determined to be unstable by: – a comparison of calculated scour and observed scour during the bridge inspection, or – an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.
1	Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic. Failure is imminent based on: – a comparison of calculated and observed scour during the bridge inspection, or – an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.
0	Bridge is scour critical. Bridge has failed and is closed to traffic

Item 113 - Scour Critical Bridges 1 digit

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Evaluations shall be made by hydraulic/geotechnical/structural engineers. Guidance on conducting a scour evaluation is included in the FHW A Technical Advisory T 5140.23 titled, "Evaluating Scour at Bridges."[1] Detailed engineering guidance is provided in the Hydraulic Engineering Circular 18 titled "Evaluating Scour at Bridges."[2] Whenever a rating factor of 2 or below is determined for this item, the rating factor for Item 60 -- Substructure and other affected items (i.e., load ratings, superstructure rating) should be revised to be consistent with the severity of observed scour and resultant damage to the bridge. A plan of action should be developed for each scour critical bridge (see FHW A Technical Advisory T 5140.23, HEC 18 and HEC 23[3]). A scour critical bridge is one with abutment or pier foundation rated as unstable due to (1) observed scour at the bridge site (rating factor of 2, 1, or 0) or (2) a scour potential as determined from a scour evaluation study (rating factor of3). It is assumed that the coding of this item has been based on an engineering evaluation, which includes consultation of the NBIS field inspection findings.

[1] FHWA Technical Advisory T 5140.23, Evaluating Scour at Bridges, dated October 28, 1991.
[2] HEC 18, Evaluating Scour at Bridges, Forth Edition.
[3] HEC 23, Bridge Scour and Stream Instability Countermeasures, Second Edition.
[4] FHWA Memorandum "Scourability of Rock Formations," dated July 19, 1991.

Figure 3.13.3-1: Designer's Coding of Item 113 – Scour Critical Bridges Form

3.13.4 Bridges Requiring Special Inspection and Maintenance Procedures

3.13.4.1 For all structures having unique or special features whose condition cannot be fully assessed through a standard visual inspection, or which require additional attention during an inspection to ensure the safety of such bridges, the Designer will prepare a Special Inspection Procedure and will submit it along with the contract documents as a design deliverable. The Special Inspection Procedure will outline the procedures and methods required to properly inspect their condition and could include the use of Non-Destructive Testing equipment, periodic measurements at identified locations, and elevation surveys to properly assess the condition of such features.

Examples of such special and unique features are:

- Cable stayed bridges: cable stays, their anchorage to the bridge and the tower, structural tower inspection.
- Segmental concrete bridges: post tensioning cables and their anchorages, sagging of the structure due to strand relaxation or deterioration.
- Bridge with settling substructures: periodic survey of elevations at piers to monitor settlement rates.

Since it is impossible to outline every potential type of unique or special feature, it is incumbent upon the Designer to consider future inspection needs if the design calls for details which are not part of the MassDOT standards as detailed in Part II of this Bridge Manual. If the Designer is not certain if a Special Inspection Procedure is required, the MassDOT Bridge Inspection Unit should be consulted as early as possible in the design process.

3.13.4.2 For those structures that have unique or special features which require special periodic maintenance to insure their satisfactory and safe operation, the Designer will prepare a Special Maintenance Procedure Manual and submit it along with the contract documents as a design deliverable. This manual will outline the maintenance work that is required, the frequency of the required maintenance, and any special procedures required to perform the work.